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REPORTS
ON
SUBGRADE SOIL STUDIES

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CONTENTS

	Page
Subgrade Soil Constants, Their Significance, and Their Application in Practice:	
Part I. The Physical Properties of Soils and Their Effect on Subgrade Performance	1
Part II. A Discussion of the Soil Constants and the Soil Identification Chart	21
Part III. Utilization of the Subgrade Soil Identification Chart	38
The Soil Profile and the Subgrade Survey	51
Procedures for Testing Soils for the Determination of the Subgrade Soil Constants	65
Graphical Solution of the Data Furnished by the Hydrometer Method of Analysis	76

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SUBGRADE SOIL CONSTANTS, THEIR SIGNIFICANCE, AND THEIR APPLICATION IN PRACTICE

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PART I: THE PHYSICAL PROPERTIES OF SOILS AND THEIR EFFECT ON SUBGRADE PERFORMANCE [1]

PRACTICAL information on the subject of subgrades is naturally divided into two classes: That which relates to the study of the soil, and that which relates to the practical utilization of the results of soil studies in the design and construction of roads. The former is of interest primarily to the subgrade testing engineer while the latter is of interest to every engineer who is now engaged in, and every engineering student who ever expects to engage in, the construction of works on the earth's surface.

The subgrade testing engineer who makes the studies must be familiar with the detailed procedures for making the subgrade surveys, the simplied subgrade soil tests, and the more elaborate Terzaghi tests. He must understand the complete significance of the various tests and the particular tests to be used for various purposes. The designing engineer requires only a superficial knowledge of the significance of the various tests and the procedures for making them. His special interest lies in the utilization of the test results to increase the stability and permanence of the structures with which he has to deal.

During the past few years the Bureau of Public Roads, by means of published reports, lectures, and exhibits, has made known to the interested public the progress of its subgrade studies. These researches have now reached a point where it is desirable to coordinate and summarize the results obtained.

The present report, of which the first part is published in this issue, consists of three major divisions: (1) A discussion of soil properties important with respect to subgrade performance, (2) the significance of the simplified soil tests for disclosing the presence of the important subgrade soil properties, and (3) the practical utilization of subgrade soil tests in practice.

The first division, which is intended primarily for the designing engineer and the engineering student, is included in Part I. In this part of the report the authors attempt in as simple a manner as possible to disclose the relation between the vehicle, the road, and the subgrade groups which have been suggested in a previous report and to discuss in a consistent order the various physical principles controlling the performance of the subgrade. An effort is made to show (*a*) that the subgrade instead of the pavement really supports the wheel load, (*b*) that the manner in which the subgrade supports the wheel load depends upon its reaction to both load and climatic changes, (*c*) that these reactions depend upon the five basic physical characteristics of soils, to wit, cohesion, internal friction, compressibility, elasticity, and capillarity, (*d*) that these physical characteristics control such important performances of subgrades as shrinkage, expansion, frost heave, the settlement of fills, sliding in cuts and lateral flow of soft undersoils, (*e*) that these physical characteristics are furnished by soil constituents easily identified in the laboratory and (*f*) that subgrades may be arranged in definite groups according to the characteristics of the soil constituents.

IMPORTANCE OF SUBGRADE SOIL CONSTANTS DISCUSSED

A subgrade soil test result may be defined as a measure of the degree in which a particular physical characteristic is exhibited when a soil is tested according to some arbitrary procedure. A subgrade soil constant may be either a test result as such or the result of a computation involving the use of several test results.

The subgrade soil constants to be employed beneficially in practice must serve to disclose the existence of those subgrade properties which exert an important influence upon the service rendered by road surfaces.

In order that subgrade soil constants may perform this service, one must have some conception of (*a*) those physical characteristics of subgrade soils which have an important bearing on the serviceability of road surfaces, (*b*) the influence exerted by the condition in which the soil exists and the character of its constituents upon the important subgrade soil properties, (*c*) the laws which control the physical characteristics possessed by subgrade soils, and (*d*) the degree to which subgrade soil constants disclose the presence of important subgrade characteristics.

Information of the character referred to is furnished by the subgrade investigations and the reports regarding them which have been published at different times in PUBLIC ROADS and elsewhere. These reports are listed in the bibliography included as part of this report.

While there is no intent to minimize in any manner the important influence exerted upon the properties of the soil by the state in which it exists, this report discusses primarily those properties characteristic of the raw constituents of soils regardless of state, and the importance of those properties with respect to road construction.

It should be remembered that the suggested subgrade groups are based upon subgrade performance. As additional information becomes available it might be desirable to subdivide certain of the groups with respect to the degree in which the subgrades possess particular properties, but the main groups are not likely to change in definition. The test constants which are being suggested as a means of identifying the members of the various groups are in a state of development and can not be considered as final. However, these constants and the scheme suggested for their use constitute the most logical method of soil identification yet disclosed by the bureau's subgrade investigations. This material is presented at this time not as a final and conclusive treatise on soil identification, but rather as a rational method by means of which the usefulness of test constants may be intelligently investigated.

Italic figures in parentheses ( ) used in this report refer to reports listed in the bibliography (page 49) which furnish the material being discussed.

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[1] Reprinted from PUBLIC ROADS, vol. 12, No. 4, June, 1931.

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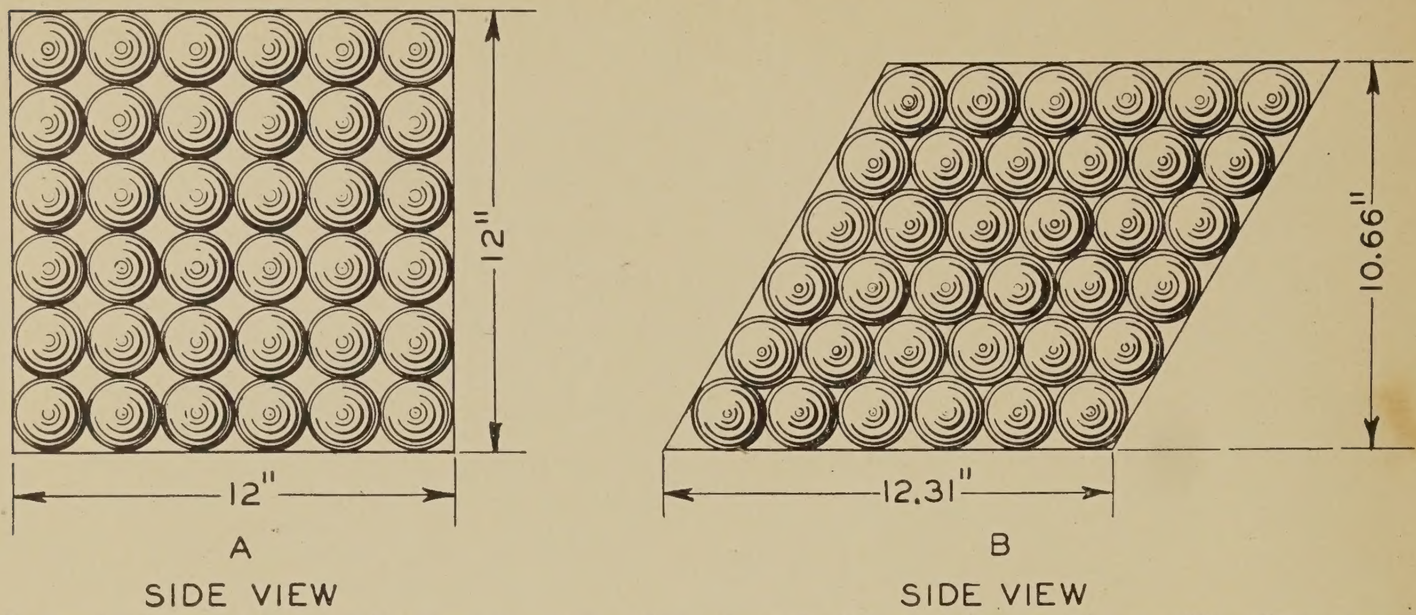


FIGURE 1.—SPHERES 2 INCHES IN DIAMETER ARRANGED IN LOOSE STATE, SIX POINTS OF CONTACT (A) AND IN MORE CONSOLIDATED STATE, EIGHT POINTS OF CONTACT (B)

Procedures for determining the different constants and for making subgrade surveys and mapping subgrade soil profiles are being prepared for publication at a later date.

BOTH SUPPORTING VALUE AND MOVEMENT OF SUBGRADE IMPORTANT IN ROAD CONSTRUCTION

In order to appreciate the importance of the subgrade in road construction one has only to remember that instead of the road surface, the ground or subgrade beneath really supports his pleasure car, truck, or other vehicle. The road surface or pavement merely distributes the vehicle weight over areas larger than those furnished by the tires alone.

Our high-class pavements, even the most substantial, when robbed of this ground support for appreciable distances, say 15 or 18 feet, break of their own weight and when unsupported for even short distances they become unable to withstand the weight of motor cars.

Thus the road surface furnishes only a smooth top on the natural ground surface and in order to accomplish this purpose two conditions must be fulfilled: The road surface must distribute the weight or impact delivered by motor vehicle wheels over an area sufficient to prevent appreciable depression of the ground beneath the pavement, and the ground beneath the pavement must be prevented from moving an amount sufficient to deform the road surface seriously. Otherwise, the road surface will fail.

When designing a highway the engineer is called upon to furnish a structure which, first, will resist the wear and tear caused by fast-moving motor wheels, second, will distribute wheel weights and impacts so as to prevent deformations which would be detrimental to either the road surface or the subgrade and, last, will resist natural forces to such an extent that their effect as manifested through subgrade movement will not be detrimental. In order to do this most economically, he must be cognizant of (1) the wheel loads and impacts to be resisted, (2) the relative ability of pavements to spread or distribute wheel loads, (3) the safe load the subgrade will support without depressing a detrimental amount, and (4) the movements likely to occur in the subgrade due either to climatic influences or to other causes. Only then is the engineer in a position to transform his road appropriations into the greatest mileage of serviceable highways.

It becomes evident, therefore, that in addition to studies of traffic weights and intensities and of pavement properties, none of which are discussed in this report, it is of great importance to investigate both the load-carrying properties of the subgrade and those soil characteristics which control subgrade movements other than those caused by vehicular loads.

VOIDS RATIO, VOLUME CHANGE, MOISTURE CONTENT, AND POROSITY EXPLAINED

It is necessary at this point to define certain terms, with full explanations of their significance. While the engineer who seeks only a general knowledge of subgrades may never have to use them in tests of his own, it is essential to an understanding of the subject that he know the precise meaning of the terms "voids ratio," "volume change," "moisture content," and "porosity," which have to do with those changes in soil state that affect the performance of subgrades.

A soil mass, or soil, as generally termed, consists of both soil particles and pores. When a soil mass, due to change in either moisture content or degree of consolidation, either increases or decreases in volume, only the void volume or the pore space is assumed to change, the volume of the soil particles remaining constant.

The density which controls in a large measure the supporting value of the soil depends upon the ratio of pore volume to either soil particle or soil mass volume. The test constants which represent either the moisture contents of soils when in particular states, or the changes in moisture content caused by changes in soil states, indicate, among other things, the density of the soil.

In order to visualize the soil states indicated by the constants and by the different degrees of soil density one must thoroughly understand the significance of the terms "voids ratio," "moisture content," and "porosity," which disclose the relation of pore volume to soil particle volume in the soil mass.

Voids ratio.—This term is defined as the ratio of the volume of voids to the volume of soil particles in a soil mass (*1*), i. e., the volume of the voids or pores per unit volume of soil particles in a soil mass.

Thus if e = voids ratio;
V_v = volume of voids;
V_s = volume of soil particles;

$$e=\frac{V_v}{V_s} \qquad (1)$$

and $e+1$ = total volume of soil mass per unit volume of soil particles in the mass.

The voids ratio, e, varies with (1) variation in degree of compaction, the number of soil particles remaining constant, (2) increase or decrease in moisture content, the number of soil particles remaining constant, and (3) increase or decrease in total number of soil particles, the volume of the soil mass remaining constant.

To illustrate the significance of the voids ratio, assume 216 spheres to be arranged as shown in Figure 1, A in a container 1 cubic foot in volume and a rectangular parallelepiped in form.

The combined volume of the voids and the spheres representing that of the soil mass equals that of the container, 1,728 cubic inches.

The volume of the spheres is given by the equation,

$$V_s=\frac{4}{3}\pi r^3\times 216=904.8 \text{ cubic inches}$$

The volume of voids is the difference between these two volumes,

$$V_v=1728-904.8=823.2 \text{ cubic inches}$$

Hence the voids ratio,

$$e=\frac{823.2}{904.8}=0.910$$

By a rearrangement of the spheres the voids ratio may be changed. This is illustrated by placing the 216 spheres in a container which is an oblique parallelepiped in form, as shown in Figure 1, B.

In this case the volume of the container is equal to the product, $12\times 12.31\times 10.66=1{,}575$ cubic inches; $V_v=1{,}575-905=670$ cubic inches; and the voids ratio, $e=\frac{670}{905}=0.740$.

Volume change.—This term is defined as the change in the volume of a soil mass due to change in the state of consolidation of the soil particles. Volume change is expressed in percentage of the volume of the soil mass either before or after change in the state of consolidation.

Thus when a soil changes from one state of consolidation indicated by a volume equal to V_1 to a different state of consolidation indicated by a volume V_2, C_1 is the volume change in percentage of the volume of the soil, V_1; and C_2 is the volume change in percentage of the volume V_2. Thus

$$C_1=\frac{V_1-V_2}{V_1}\times 100$$

$$C_2=\frac{V_1-V_2}{V_2}\times 100$$

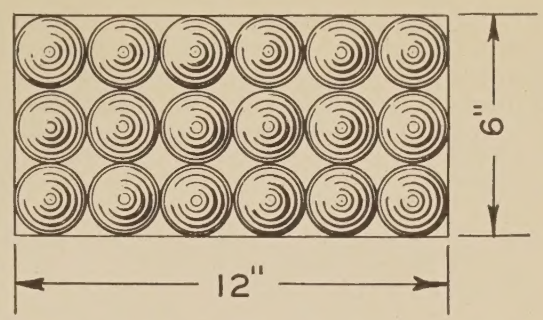


Figure 2.—108 Spheres Arranged With Six Points of Contact on Each Sphere, Voids Ratio Equal to that of the 216 Spheres Shown in Figure 1, A

If the spheres shown in Figure 1, A are assumed to represent a state of consolidation indicated by V_1 and those shown in Figure 1, B are assumed to represent a state of consolidation indicated by V_2, the volume change obtained by changing the arrangement of the spheres from that shown in Figure 1, A to that shown in Figure 1, B is given by the formula,

$$C_2=\frac{V_1-V_2}{V_2}\times 100=\frac{1{,}728-1{,}575}{1{,}575}\times 100=9.7 \text{ per cent}$$

This method of computing volume change may be used whenever the volume of the soil mass but not the number of soil particles changes.

When a soil changes from any wet state indicated by a volume equal to V to the thoroughly dried state indicated by the volume equal to V_o, C is defined as the volume change in percentage of the volume of the soil in the wet state and C_o is defined as the volume change in percentage of the volume of the soil in the dry state. Thus

$$C=\frac{V-V_o}{V}\times 100; \text{ and}$$

$$C_o=\frac{V-V_o}{V_o}\times 100 \qquad (2)$$

Thus, for instance, the volume change which occurs when soil cakes in the laboratory are changed from the wet to the dry state, as in shrinkage tests, is given by this formula.

In case, however, one desires to compute the volume change when both the degree of consolidation and the number of soil particles change he must of necessity employ the voids ratio, which is expressed in unit values.

Assume, for instance, that the arrangement of the spheres shown in Figure 1, A remains the same but that the number of spheres is reduced to 108 as shown in Figure 2. The volume of the container is reduced by one-half but the voids ratio of necessity remains the same,

$$e=\frac{864-452.4}{452.4}=0.910$$

Let e_1 equal the voids ratio possessed by the 108 spheres shown in Figure 2, state 1, and e_2 equal the voids ratio of the 216 spheres shown in Figure 1, B, state 2. Then, if the degree of consolidation of the spheres is changed from state 1 (figs. 1, A or 2) to state

2 (fig. 1, B), the volume change of the container or soil mass per unit volume of soil particle, expressed in percentage of the volume of the container per unit volume of soil particle, state 2, is given by the expression,

$$C_2=\frac{(1+e_1)-(1+e_2)}{1+e_2}\times 100$$

$$=\frac{e_1-e_2}{1+e_2}\times 100 \qquad (3)$$

$$=\frac{0.910-0.740}{1+0.740}\times 100=9.8 \text{ per cent}$$

which substantially agrees with the value of C_2 obtained above, the difference being due to the lack of decimal places in the values of e.

Moisture content.—The moisture content, w, is defined as the weight of moisture in the soil in percentage of the weight of the soil particles.

Thus, if M_w is defined as the weight of the soil moisture in grams and W_o as the weight of the soil particles (weight of thoroughly dry sample) in grams,

$$w=\frac{M_w}{W_o}\times 100 \qquad (4)$$

To determine the moisture content possessed by a soil, the soil sample is weighed first wet and then dry. Hence, if W is defined as the weight of the wet sample (weight of soil particles + weight of moisture) and W_o as the weight of the dried soil sample,

$$M_w=W-W_o \qquad (5)$$

and

$$w=\frac{W-W_o}{W_o}\times 100 \qquad (6)$$

Because of the fact that 1 cubic centimeter of water weighs 1 gram, the weight of the water in grams, M_w, is also the volume of water in cubic centimeters. The volume of the soil particles in cubic centimeters equals the weight of the soil particles in grams divided by the specific gravity of the soil particles. Thus, if

V_s = volume of the soil particles in cubic centimeters;

and

G = specific gravity of the soil particles; then

$$V_s=\frac{W_o}{G} \qquad (7)$$

Consequently, w_v, the moisture content of the soil in percentage of the volume of the soil particles is given by the equation,

$$w_v=\frac{M_w}{\frac{W_o}{G}}\times 100$$

$$=\frac{W-W_o}{W_o}\times 100\times G$$

$$=wG \qquad (8)$$

Since w_v, the moisture content in percentage of the volume of the soil particles is equal to the void volume in percentage of the volume of soil particles, when the voids are completely filled with water, we have

$$w_v=e\times 100 \qquad (9)$$

and by substitution of wG for w_v,

$$e=\frac{wG}{100} \qquad (10)$$

Thus, if a soil sample weighs 30 grams when wet, 25 grams when dry, and the soil particles have a specific gravity of 2.5,

$$M_w=W-W_o=30-25=5 \text{ grams};$$

$$w=\frac{M_w}{W_o}\times 100=\frac{5\times 100}{25}=20 \text{ per cent};$$

$$w_v=w\times G=20\times 2.5=50 \text{ per cent};$$

$$e=\frac{w_v}{100}=0.5.$$

Porosity.—The porosity, P, is defined as the volume of the voids or pores in a soil mass in percentage of the volume of the soil mass (volume of soil particles + volume of the voids). Its value is given by the formula,

$$P=\frac{V_v}{V_v+V_s}\times 100=\frac{e}{1+e}\times 100 \qquad (11)$$

Thus, for the soil sample referred to above,

$$P=\frac{e}{1+e}\times 100=\frac{0.5\times 100}{1+0.5}=33.3 \text{ per cent.}$$

For the spheres shown in Figure 1, A,

$$P=\frac{0.91}{1+0.91}\times 100=47.6 \text{ per cent}$$

and for the spheres shown in Figure 1, B,

$$P=\frac{0.740}{1+0.740}\times 100=\frac{74}{1.74}=42.5 \text{ per cent.}$$

It has been shown [2] that the porosity of spheres of equal size when in the densest possible state equals approximately 26 per cent and the corresponding voids ratio approximately 0.35.

INFORMATION FURNISHED BY SUPPORTING VALUE TESTS LIMITED IN SCOPE

A scrutiny of reports on experiments dealing with the subject discloses a diversity of opinion as to the best manner not only of measuring but also of expressing the magnitude or efficiency of the support furnished to the road by the subgrade.

Thus the "comparative bearing value" of soils studied in the early investigations of the Bureau of Public Roads (*2*), the "modulus of subgrade reaction" used by H. M. Westergaard (*3*) in his discussions, and the "consistency" of soils investigated by Charles Terzaghi (*1*, *4*) are all indicative of supporting value and yet differ widely in both significance and scope.

The nature of the comparative bearing value test and the type of information furnished by it are illustrated in Figure 3. It is, in brief, a simple load-deformation, or load-penetration test of a small soil sample held in a container. A curve is drawn which shows

[2] Taylor and Thompson, Concrete, Plain and Reinforced, second edition, 1912, pp. 168–170.

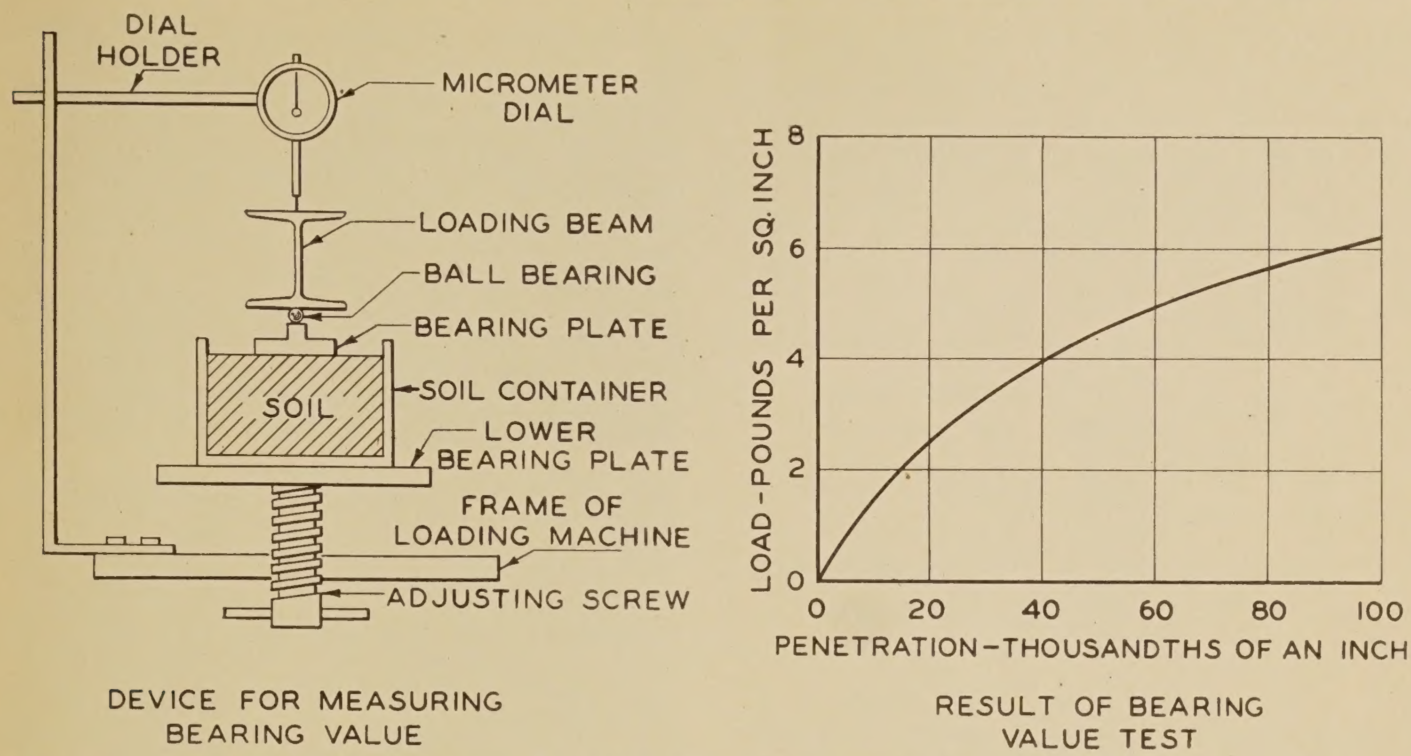


Figure 3.—Apparatus and Type of Results Furnished by Bearing Value Test

how much a given load will deform a given sample. Thus, according to Figure 3, a load of 4 pounds per square inch causes the bearing plate to penetrate a particular soil sample 0.041 inch. For comparative purposes, the bearing value of the soil was assumed to be the load in pounds per square inch required to produce a penetration of 0.1 inch.

Doctor Westergaard's modulus of subgrade reaction may be defined as the load in pounds per square inch required to deform a perfectly elastic subgrade 1 inch. Thus a perfectly uniform and perfectly elastic subgrade which will deform 0.01 inch for each one-half pound per square inch of pressure applied, has a modulus of subgrade reaction equal to 50.

The Terzaghi consistency test is performed on a cylindrical soil sample in the following manner. The sample is mounted in a loading machine equipped with a micrometer dial for measuring deformations, as shown in Figure 4. Load is applied slowly to a predetermined magnitude, and the sample is allowed to deform under this load until a state of equilibrium is reached. During this period of constant load the deformation is recorded as a function of the time. The load is then removed, and applied again to a greater magnitude. The deformation as a function of time is recorded for this load; the load is removed and applied a third time, until a point is reached (the yield point) where deformation is continuous without increase in load. The curve in Figure 4, center, shows the type of load-deformation curve which results from such a test. The curve at the bottom shows deformation plotted as a function of time.

It is evident that this test takes into consideration not only the load-deformation relation which the comparative bearing value test was designed to give, and the elastic rebound assumed by Westergaard, but also the effect of time on the deformation.

Even the consistency test, however, fails to supply complete information on subgrade support which should also include a knowledge of:

1. The deformation of the soil as influenced by (*a*) the magnitude of applied load, (*b*) the size and shape of the loaded area, and (*c*) a surcharge adjacent to the loaded area.

2. The relative amounts of the deformation due to (*a*) lateral displacement of the loaded soil and (*b*) compression of the under soil without lateral displacement.

3. The tendency of the soil to remain compressed or rebound upon the removal of load.

Direct bearing value tests, both in time and effort required, are generally too elaborate for use as routine tests for subgrade soils. Consequently, instead of direct tests of supporting value comparatively simple tests are used in the subgrade investigations to disclose the presence of subgrade characteristics indicative of three properties which either singly or in combination control the many types of deflection produced in soils by loading.

These properties af soils may be defined as follows:

1. Stability, the property of resisting lateral flow when loaded.

2. Compressibility, the property of compressing vertically under load without lateral movement and with a proportional decrease in air or moisture content.

3. Elasticity, the property of deforming under load and rebounding upon the removal of load without changing mositure content.

Figure 5, top, illustrates the character of deformation produced by loss of stability in soils. Here the load displaces the soil laterally. The deformation due to the compressibility of soils, illustrated in Figure 5, center, consists entirely of a more or less permanent consolidation of soil particles in the vertical direction. Figure 5, bottom, illustrates the rebound upon the removal of load in elastic soils.

Loss of stability may cause fills to slide, clay to work up into the interstices of base courses and rutting to occur in flexible road surfaces. Examples of loss of stability are illustrated in Figures 6 and 7.

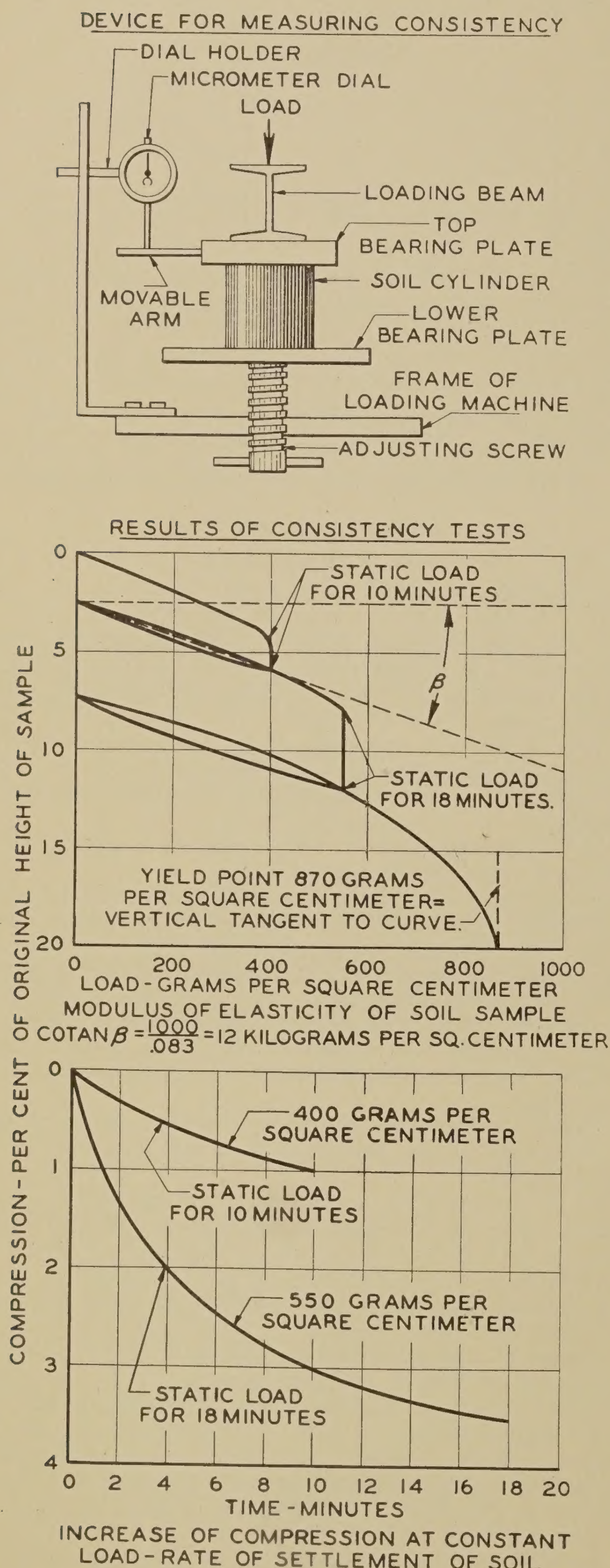


Figure 4.—Apparatus and Type of Results Furnished by Consistency Tests

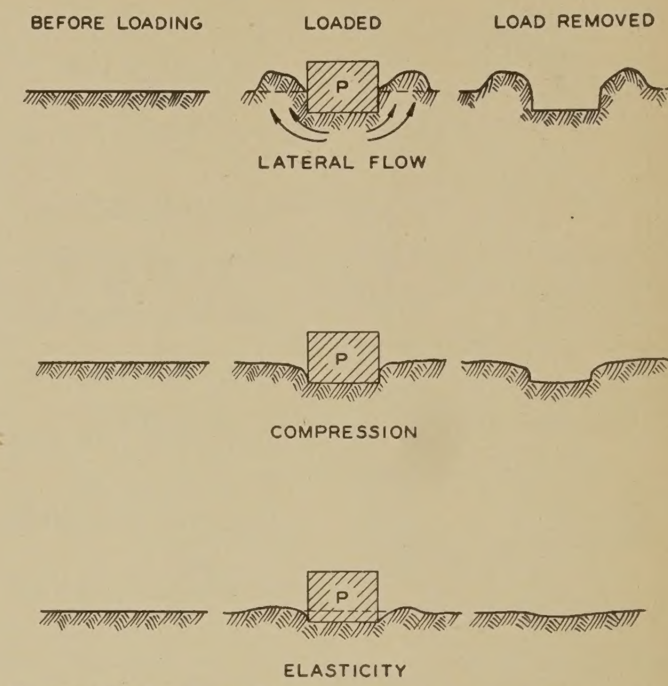


Figure 5.—Diagram Illustrating Properties on Which Reaction Between Soil and Load Depend

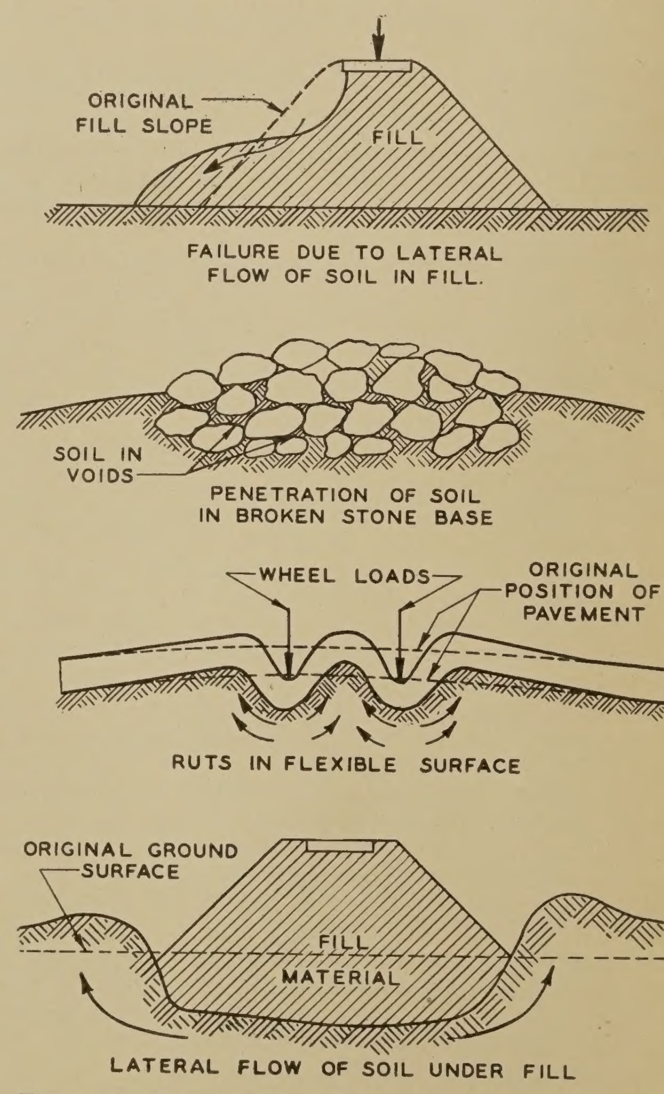


Figure 6.—Types of Road Failure Caused by Lateral Flow of the Subgrade Soil

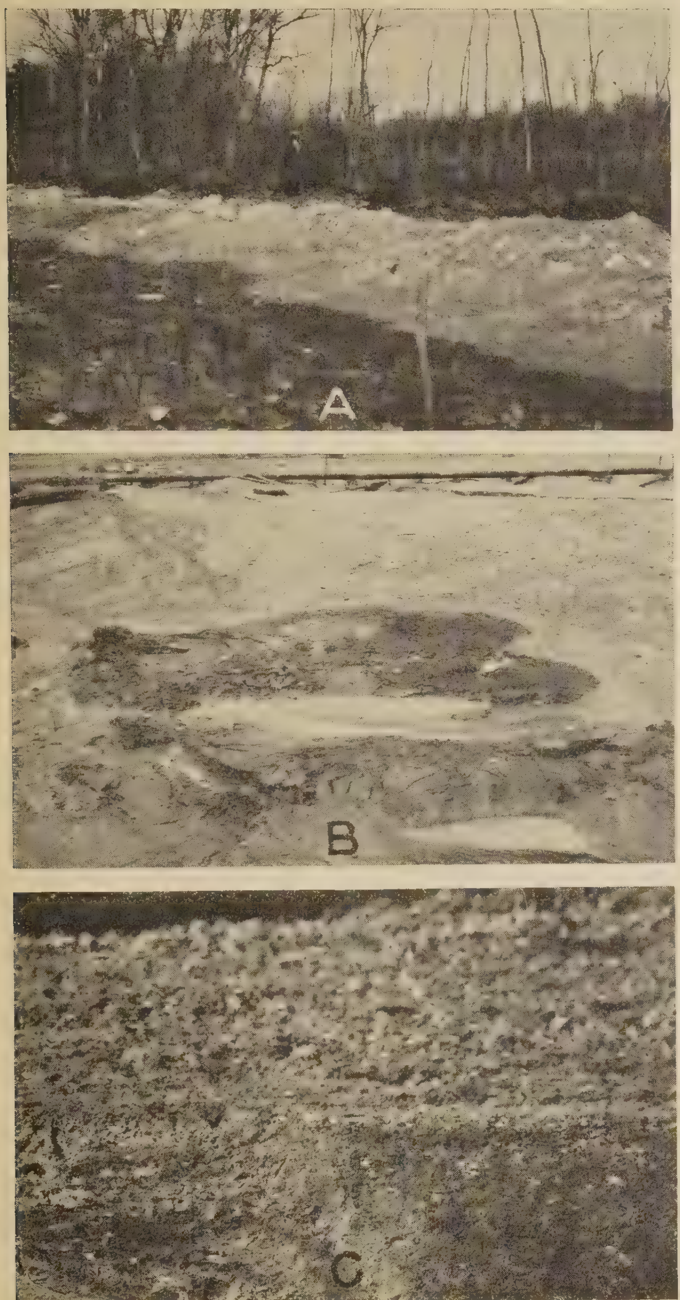


FIGURE 7.—EFFECTS OF LATERAL FLOW IN SOILS. A.—RIDGE OF MUCK SOIL FORCED UP BY LATERAL FLOW OF SOFT UNDERSOIL DUE TO WEIGHT OF FILL MATERIAL. DITCH OPENED AFTER UPHEAVAL. B.—RIVER BOTTOM FORCED UPWARD DUE TO LATERAL FLOW UNDER HYDRAULIC FILL. C.—SLIDE IN FACE OF CUT

STABILITY OF SOILS CONTROLLED BY THE COMBINED EFFECT OF INTERNAL FRICTION AND COHESION

Stability depends upon the shear strength which in turn depends upon the combined effect of the two mechanical properties of soils, internal friction and cohesion.

The magnitude of the cohesion possessed by a soil is independent of the outside pressure acting on the soil. It depends upon the stickiness of the soil grains or their resistance to being pulled apart and thus consists of the true cohesion of soil particles combined with that furnished by the molecular attraction of water (*5, 6*). The stickiness of the clay in sand-clay roads and that of bituminous materials in black-top pavements represent true cohesion of materials. The very stable support furnished racing automobiles by beach sands when wet compared with the low stability of similar sands when dry serves to illustrate the importance of that portion of the total cohesion furnished by the molecular attration of water.

Internal friction, the magnitude of which increases in direct proportion to the pressure exerted upon the soil, depends upon the resistance of the soil grains to sliding over each other (*6*). It is defined as the angle whose tangent is the ratio between the resistance offered to sliding along any plane in the soil and the component of the applied force acting normal to that plane. The sand in sand-clay roads and the mineral aggregate in bituminous surfaces furnish the internal friction.

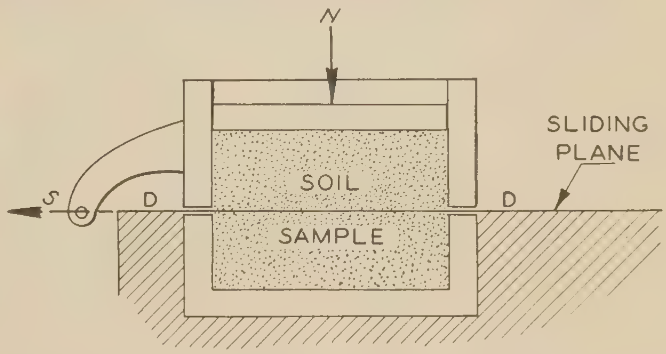


FIGURE 8.—SHEAR RESISTANCE OF SOILS AS RELATED TO INTERNAL FRICTION AND COHESION. BASIC PRINCIPLE OF APPARATUS TO DETERMINE MAGNITUDE OF THESE TWO MECHANICAL PROPERTIES

Figure 8 illustrates the influence exerted by both the cohesion and the internal friction upon the shear strength of soils.

The shear resistance is represented by the vector S. Let

N = Pressure acting on soil sample normal to the sliding plane;
c = Cohesion;
= S, when $N=o$;
ϕ = Angle of internal friction.

Then

$N \tan \phi$ = Frictional resistance to sliding;
$S = N \tan \phi + c$;

and

$$\phi = \text{arc} \tan \frac{S-c}{N}.$$

Figure 9 illustrates the conditions required for stability in homogeneous soils.

Let DD be any plane in the soil making the angle a with the horizontal,

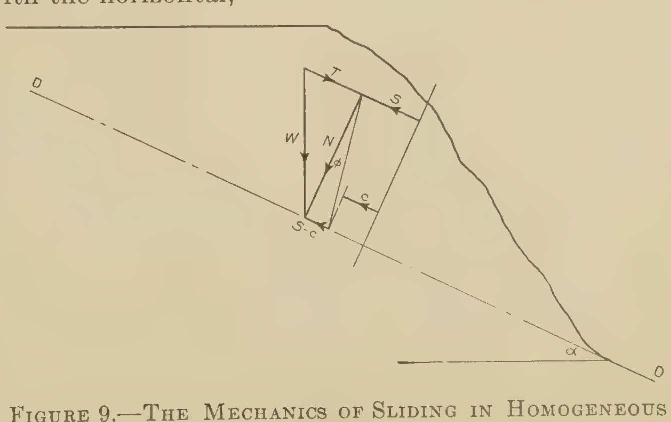


FIGURE 9.—THE MECHANICS OF SLIDING IN HOMOGENEOUS SOILS

W = unit weight of soil;
N = component of W normal to DD;
$= W \cos \alpha$;

then

T = force productive of sliding;
$= W \sin \alpha$;

and

S = shear resistance of soil;
$= N \tan \phi + c$.

Sliding occurs when T exceeds S. Therefore the requirement for stability is that $W \sin \alpha$ be less than $W \cos \alpha \tan \phi + c$.

Based upon this theoretical conception of stability, formulas have been developed by means of which may be determined (*a*) the influence of cohesion, internal friction, width of loaded area, and load adjacent to the loaded area upon the stability of subgrades (*7*) and (*b*) the influence of cohesion, internal friction and slope upon the critical height of fills (*8*).

The formula for computing the supporting value of soils was derived on the assumption that the loaded area was very long compared with its width. This assumption does not satisfy the condition produced by a wheel load upon the pavement, which is not susceptible of simple mathematical treatment. The analysis based upon a long loaded area illustrates the relative influence exerted by cohesion and internal friction upon the stability of soils.

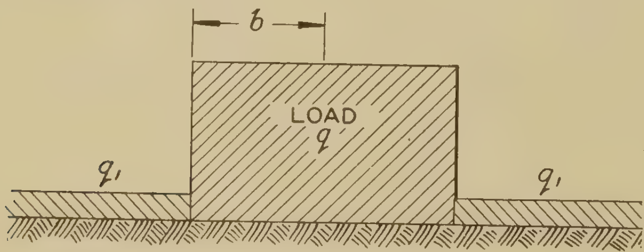


FIGURE 10.—DIAGRAM FOR ANALYSIS OF SUPPORTING POWER OF SOILS

Space does not permit derivation of the formula in this article (*7*). Figure 10 illustrates the method employed.

Let

q = load per unit area;
q_1 = surcharge adjacent to loaded area in same units;
b = ½ width of loaded area;
s = unit weight of soil;
c = cohesion (force per unit area);
ϕ = angle of internal friction;
$\beta = 45° - \frac{\phi}{2}$.

The load q, defined as the supporting value of the soil under the given conditions of cohesion, internal friction, width of loaded area, and surcharge, is given by the formula

$$q = \frac{q_1}{\tan^4\beta} + \frac{bs}{2\tan\beta}\left[\frac{1}{\tan^4\beta} - 1\right] + \frac{2c}{\tan\beta \sin^2\beta} \quad (12)$$

Résal's formula for computing the critical heights of cuts or fills (see fig. 11) gives only approximate results. It may be stated as follows:

Let

i = angle of inclination of fill;
ϕ = angle of internal friction;
Δ = unit weight of soil;
c = cohesion;

Then h_1, the critical height of fill above which sliding will occur is given by the formula,

$$h_1 = \frac{c \sin i \cos \phi}{\Delta \sin^2 \frac{i - \phi}{2}} \quad (13)$$

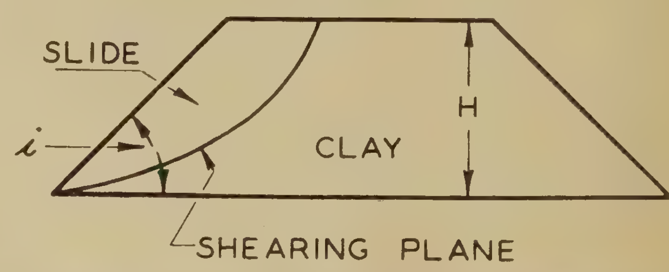


FIGURE 11.—DIAGRAM ILLUSTRATING RÉSAL'S FORMULA FOR COMPUTING THE CRITICAL HEIGHT OF CUTS OR FILLS

Results of computations made according to the formula of Figure 10 are shown in Table 1, and those made according to Résal's formula, Figure 11, are shown in Table 2.

TABLE 1.—*Influence of cohesion, internal friction, width of loaded area and load adjacent to loaded area upon stability of subgrades*

Soil type	Cohesion, c	Angle of internal friction, ϕ	Supporting value, q, pounds per square foot		
			$q_1=0$ $s=100$ pounds per cubic foot $b=0.71$-foot	$q_1=100$ $s=100$ pounds per cubic foot $b=0.71$-foot	$q_1=0$ $s=100$ pounds per cubic foot $b=7.10$ feet
	Pounds per square foot	°			
Clay, almost liquid	100	0	400	500	400
Clay, very soft	200	2	860	980	910
Clay, soft	400	4	1,860	1,990	1,960
Clay, fairly stiff	1,000	6	4,980	5,130	5,170
Clay, very stiff	2,000	12	12,540	12,770	13,060
Silts,[1] wet	0	10	40	240	430
Sands, dry	0	34	770	2,020	7,680
Sand-gravel mixtures, cemented[2]	1,000	34	17,840	19,090	24,750

[1] In silty soils the angle of internal friction may vary between 10° and 30° but the cohesion may be almost 0.

[2] In properly graded soils, depending upon the extent of their compaction, the angle of internal friction may exceed 34° but the cohesion may be considerably less than 1,000.

TABLE 2.—*Critical heights of slope in cuts and fills, computed from Résal's formula*

Soil type	Slope of cut or fill	Angle of slope, i	Weight of soil, Δ	Cohesion of soil,[1] c	Angle of internal friction, ϕ	Critical height of fill, h_1
		° ′	*Pounds per cubic foot*	*Pounds per square foot*	*Degrees*	*Feet*
Very soft clay	½:1	63 26	80	200	2	9
	1:1	45 00				13
	2:1	26 34				25
	4:1	14 02				55
Medium clay	½:1	63 26	90	1,000	6	43
	1:1	45 00				70
	2:1	26 34				155
	4:1	14 02				546
Stiff clay	½:1	53 26	100	1,500	8	61
	1:1	45 00				104
	2:1	26 34				255
	4:1	14 02				1,300
Good sand clays	½:1	63 26	110	1,000	34	104
	1:1	45 00				580
	2:1	26 34				Unlimited.
	4:1	14 02				Unlimited.
Silty clays	½:1	63 26	100	200	14	10
	1:1	45 00				20
	2:1	26 34				72
	4:1	14 02				Unlimited.

[1] Materials having no cohesion such as sands, silts, etc., have no critical heights.

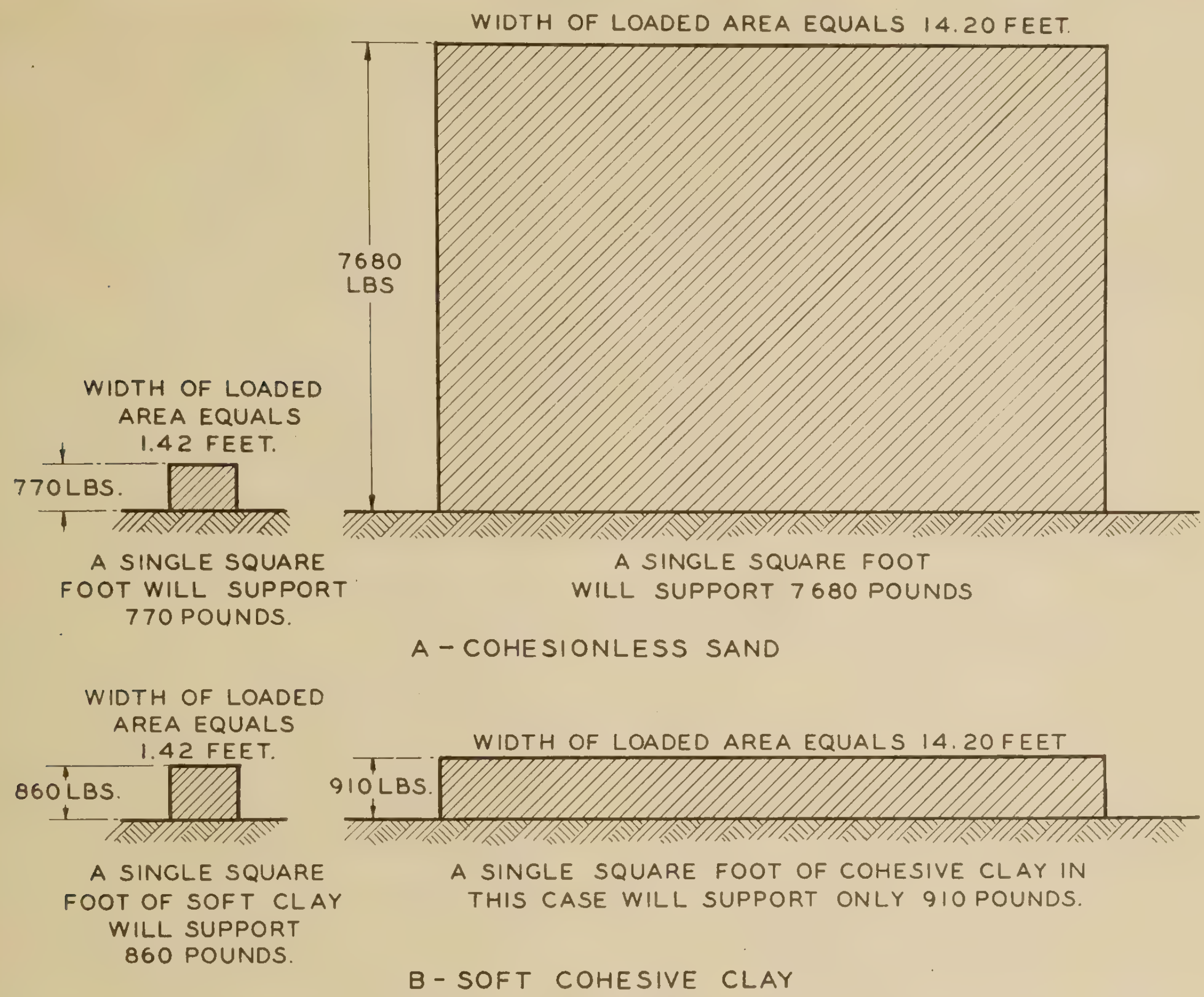


Figure 12.—Relative Effect on Supporting Power of Soils of Increasing the Area Under Load

These two formulas also serve to furnish some indication of the relative influence exerted on both the stability of subgrades and the critical height of fills by variations in the angle of internal friction, the cohesion remaining constant, or vice veras. Thus, when the cohesion equals 400 pounds per square foot, b equals 0.71 foot, and q_1 equals 0, the supporting value of the subgrade will equal either 2,550 or 7,600 pounds per square foot, depending on whether the angle of internal friction equals 12° or 34°. When the angle of internal friction equals 34° the supporting value of the subgrade will equal either 4,180 or 17,840 pounds per square foot, depending on whether the cohesion equals 200 or 1,000 pounds per square foot. Likewise, the critical height of fills with a slope of 1 to 1, a weight of soil, Δ, of 100 pounds per cubic foot, and a cohesion of 400 pounds per square foot will be either 21 or 34 feet, depending on whether the angle of internal friction equals 2° or 12°; and when the angle of internal friction equals 6° the critical height will equal 13 or 63 feet, depending on whether the cohesion equals 200 or 1,000 pounds per square foot.

These examples show not only that stability depends upon both the internal friction and the cohesion of the soil, but also that the manner in which stability is influenced by such factors as the size of the loaded area, etc., differs widely depending on whether the stability is furnished principally by internal friction or cohesion. Thus:

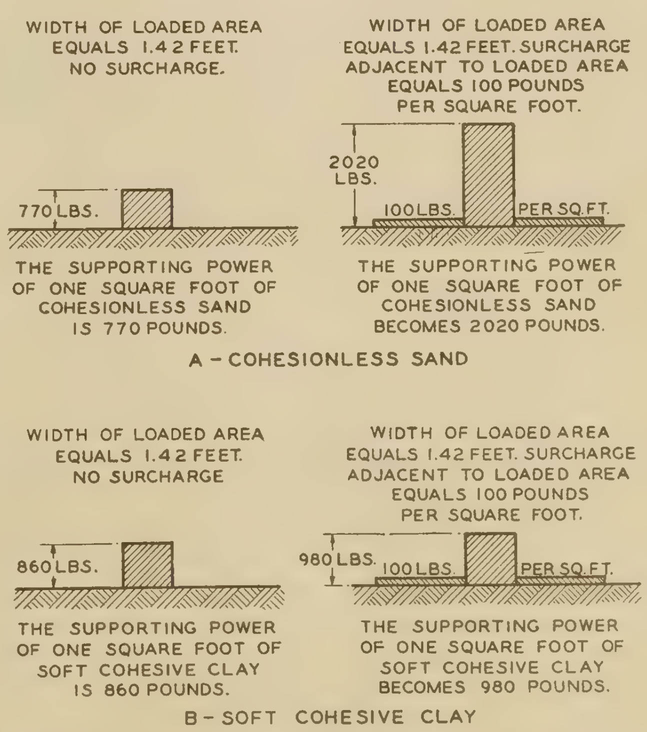


Figure 13.—Relative Effect on Supporting Power of Soils of Uniformly Loading the Soil Adjacent to the Loaded Area

1. Increasing the width of the loaded area and also surcharging the soil with a load adjacent to the loaded area increases the unit support very appreciably, as shown in Table 1 and Figures 12 and 13, when the stability of the subgrades is furnished principally by internal friction instead of cohesion. The increases in the unit support of cohesionless soils due to increase in width of loaded area and to the surcharging just noted are, according to unpublished data furnished by Dr. Charles Terzaghi, much greater when the loaded area is long and narrow than when it is square or circular.

2. Increasing the width of the loaded area or surcharging the soil with a load adjacent to the loaded area does not increase the unit support very appreciably when the stability of the subgrade is furnished principally by cohesion instead of internal friction.

3. The safe angle of repose of fill material is independent of the height of the fill only when the fill consists of cohesionless materials.

Thus one sees how a conception of the effect of the relative amounts of cohesion and internal friction possessed by the soil is more enlightening with respect to the design of preventive measures than merely a knowledge of stability or the combined effect of these two properties. Figure 14 shows a method of preventing sliding by terracing the faces of a cut made in clay.

Figure 14.—Terracing of Faces of Railway Cut in Clay, Between Washington and Baltimore. Surface Drainage is Provided for on Each Terrace. One of the Methods Used to Prevent Sliding in Cuts

RECOGNITION OF COMPRESSIBILITY AND ELASTICITY IN SUBGRADES IMPORTANT

Consolidating the subgrade serves to increase its density and decrease its permeability[3] (*1, 7, 9, 10*) and consequently is likely to prove highly beneficial. The degree of consolidation obtainable and the nature of the results, whether beneficial or detrimental, which will be obtained by attempted consolidation depend upon whether the subgrade soils are of the compressible or of the elastic type; that is, whether in the absence of change in moisture content they will remain consolidated or will rebound upon the removal of load. To remain consolidated after the removal of load, in the absence of free water, soil grains must either lack any spongy or elastic property which tends to push them apart or possess cohesion in amount sufficient to overcome such a tendency.

A small sponge and a wad of cotton will serve to illustrate very effectively how the elastic subgrades differ in performance from the compressible subgrades. If, for instance, they are both thoroughly wetted and then compressed by hand the cotton, representing the compressible soil, remains in the compressed state after the removal of the compressing force, because of capillary tension acting on the surface of the cotton—the same force which stabilized the beach sand referred to above. The capillary tension in this case equals at least the pressure exerted by the hand to compress the

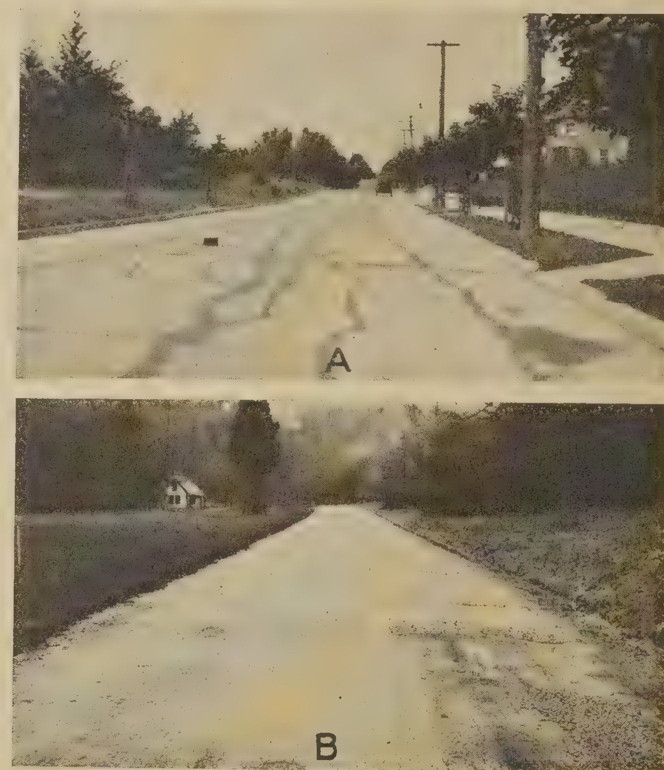


Figure 15.—Detrimental Effects of Elasticity of Subgrade Soil. A.—Type of Cracking Likely to Occur in Pavements Laid on Improperly Prepared Elastic Subgrades. Note Cracks Reflecting Ruts in Subgrade. B.—Alligator Hide Cracking in Macadam Road Surface

cotton and also any tendency possessed by the cotton strands to separate, otherwise the cotton would have expanded. The sponge, representing the elastic subgrades, expands to almost its wet volume upon the removal of the compressing force. This occurs because the tendency of the sponge fabric to expand greatly exceeds the capillary force acting upon the sponge surface. Soaking the sponge prior to compression in a glue whose molecular cohesion exceeds that of water in sufficient amount serves to prevent the elastic rebound upon the removal of pressure.

Either a macadam or a concrete surface may be seriously damaged by attempts to consolidate elastic subgrades before pavement construction. After the thorough rolling which benefits the compressible subgrades, a subgrade of the elastic type, if it possesses cohesion, is likely to retain a certain degree of compaction. A slight wetting under these conditions, such as is furnished by freshly deposited concrete, may cause a nonuniform rebound of the subgrade. This, combined with water loss from the concrete due to absorption by the soil is likely to cause pavements to crack excessively (fig. 15, A) during the setting period of the concrete.

Movements of heavy material trucks and mixing apparatus adjacent to pavements laid on elastic co-

[3] Permeability is defined as the rate at which gravitational water is transmitted by soils. It depends upon both the hydraulic gradient and the size and number of the soil pores. It varies as the square of the effective diameter of the soil grains. It is expressed as the coefficient of permeability which is designated as k and equals the velocity in centimeters per second under a hydraulic gradient of 1.

hesionless subgrades may cause distortions of the soil supporting the freshly laid concrete sufficient to produce pavement cracking. Cracking of this character may remain in microscopic form for an appreciable period of time. Except during the setting period of the concrete, elastic subgrades are not likely to be detrimental to concrete pavements.

The presence of elasticity in subgrades may prevent macadam pavements from acquiring adequate bond during construction and from retaining it subsequently. Under these conditions macadams may develop "alligator hide" (fig. 15, B) cracking, through which water may pass and cause the subgrade soil to soften and to penetrate the voids of the macadam, thus causing the surface to fail.

Figure 16.—Fill Being Constructed of Material Consisting of Clods

DEGREE AND RATE OF COMPRESSION DISCUSSED

The manner in which soils may compress depends to a large extent upon their moisture contents. Those soils whose voids contain air may compress because of either a compression of the entrapped air or the escape of the air from the soil pores. In this case the rate of consolidation depends upon such factors as the resistance of clods (fig. 16) to crushing and can not be computed. The force required to break clods differing in degree of dryness can, however, be investigated in the laboratory.

Soils in the plastic state (fig. 17) or those whose voids are filled with moisture, may consolidate vertically without flowing laterally only when water escapes from the soil pores. Thus foundations supporting buildings and other structures adequately for years settle suddenly when new excavations permit water to escape from the loaded soil supporting the foundations.

In this case the speed of soil consolidation for equal external pressures applied depends primarily upon the permeability of the soil mass. In fact within certain limits it varies directly with the coefficient of permeability of the soil. Since the coefficients of permeability of the soil constituents may vary through a wide range, as shown in Table 3, the rates at which individual soils consolidate may be widely different.

Figure 18 shows how the data furnished by the Terzaghi compression test may be employed to indicate both the amount and the rate of fill settlement. This test has been discussed previously in Public Roads by Dr. Charles Terzaghi (*1*).

Water pressed from the soil pat (fig. 18, A) by the weighted piston, passes through the porous stones above and below the pat and escapes from the overflow orifices *a* and *b*. The relation between the voids ratio of the soil and the compressing force is expressed as the load-compression curve, Figure 18, B. The data for constructing the load-compression curve are obtained by applying the compressing force in magnitudes equal approximately to 0, 0.5, 1.5, and 3.0 kilograms per square centimeter and observing the voids ratio produced by each load when applied, until further increase in the deformation of the soil ceases. Consequently the load-compression curve discloses the minimum voids ratio or the maximum density of the soil likely to be produced by loads of given magnitude. Data for constructing the expansion curve (fig. 18, B) are obtained when the load is changed successively from approximately 3.0 to 1.5, 0.5, and 0.0 kilograms per square centimeter and water is allowed to enter the sample. The data for constructing the time-compression curve are furnished by observing the times corresponding to the deformations produced by the individual load increments.

Table 3.—*Coefficients of permeability of soil constituents under pressure of 1.5 kilograms per square centimeter*

Soil	Coefficient of permeability
	Centimeter per second
Potomac River sand, 20–100 mesh	18.96×10^{-4}
Mica, 20–100 mesh	0.128×10^{-4}
Rock Creek silt	0.00096×10^{-4}
Diatoms	0.048×10^{-4}
Clay	0.00011×10^{-4}
Peat (Minn.)	0.785×10^{-4}

Figure 17.—Settlement of Road at Bridge Approach Due Primarily to the Consolidation of the Thoroughly Saturated Undersoil

The time-compression curve (fig. 18, C) shows the relation between (*a*) the degree of compression of the soil, expressed as a percentage of the total compression occurring in a very long loaded interval, and (*b*) the length of the loaded interval expressed in minutes, the magnitude of the load remaining constant.

In order to demonstrate, by analogy, the method of estimating both the magnitude and the rate of compression caused by a hydraulic fill constructed on inundated river bottom land as illustrated in Figure 18, D, it must be assumed first that the compression is direct and no lateral thrust is involved; second, that both the fill material on top and the hard compact sand beneath are more permeable than the soft soil layer; third, that the moisture is free to pass through the underlayer of

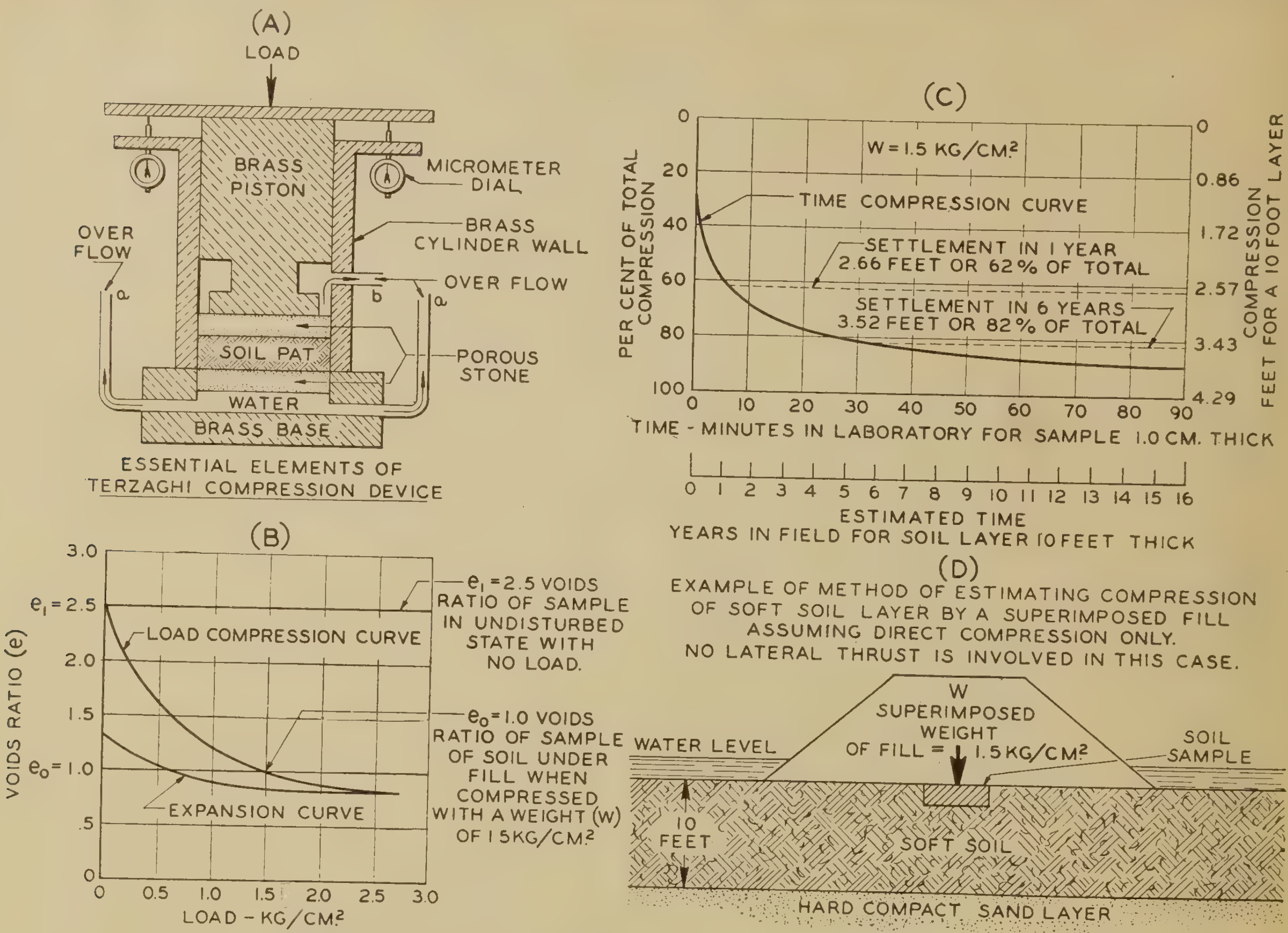


FIGURE 18.—DATA FURNISHED BY THE TERZAGHI COMPRESSION TEST

sand, and last, that the moisture content throughout the thickness of the soft soil layer is uniform.

Under these conditions the soft soil layer in the river bottom (fig. 18, D) is subjected to compression similar to that acting on the soil pat (fig. 18, A). Thus the weight of the fill material corresponds to the compressing force, and the fill material on top and the compact sand beneath the soft soil layer correspond to the porous stones. (Fig. 18, A). In addition let the load-compression curve (fig. 18, B) and the time-compression curve (fig. 18, C) be assumed to represent tests performed upon an undisturbed sample of the soft river bottom soil, as indicated in Figure 18, D.

Under the weight of the fill material (1½ kilograms per square centimeter), the undersoil will, since the assumed conditions are the same as those existing in the laboratory, compress from a voids ratio $e_1 = 2.5$ (original undisturbed state) to a voids ratio $e_0 = 1$. The ratio of soil thickness after compression to soil thickness before compression equals the ratio of soil volume (soil particles plus voids) after compression to the soil volume before compression, or $\frac{1+e_0}{1+e_1}$. Consequently the soil layer 10 feet thick will compress to a layer whose thickness is given by the expression

$$\frac{1+e_0}{1+e_1} \times 10 = 5.71 \text{ feet.}$$

The time in years required for different stages of settlement is computed from the time-compression curve on the assumption that the time required for two soi layers to compress in equal degree varies as the squares of the thickness of the layers.

Accordingly Figure 18 informs us (*a*) that the fill will settle 4.29 feet and (*b*) that 2.66 feet of this amount will occur during the first year and that an additional settlement of 0.86 foot will occur during the succeeding five years.

CAPILLARITY THE IMPORTANT AGENT CAUSING CHANGES OF WEATHER TO BE REFLECTED IN SUBGRADE MOVEMENTS

The more important subgrade movements due to climatic influences are (*a*) expansion of the soil occurring with an increase in moisture content (*b*) shrinkage of the soil occurring with a decrease in moisture content, and (*c*) heaving of the soil during frost.

These occurrences depend upon the physical phenomenon capillarity, which, so far as subgrades are concerned, is defined as the ability of soils to transmit moisture in a finely divided state in all directions in spite of both the direction in which gravity acts and the force of gravity.

Capillary action is illustrated in Figure 19. Capillarity draws the liquid up through the cheesecloth wicks, over the edge of the container A, and down to the ends of the wicks on the outside of the container. This merely moistens the wick on the left whose outside end is just about on a level with the surface of the liquid in the container. In addition to becoming moist, the wick on the right, the outside end of which is located appreciably

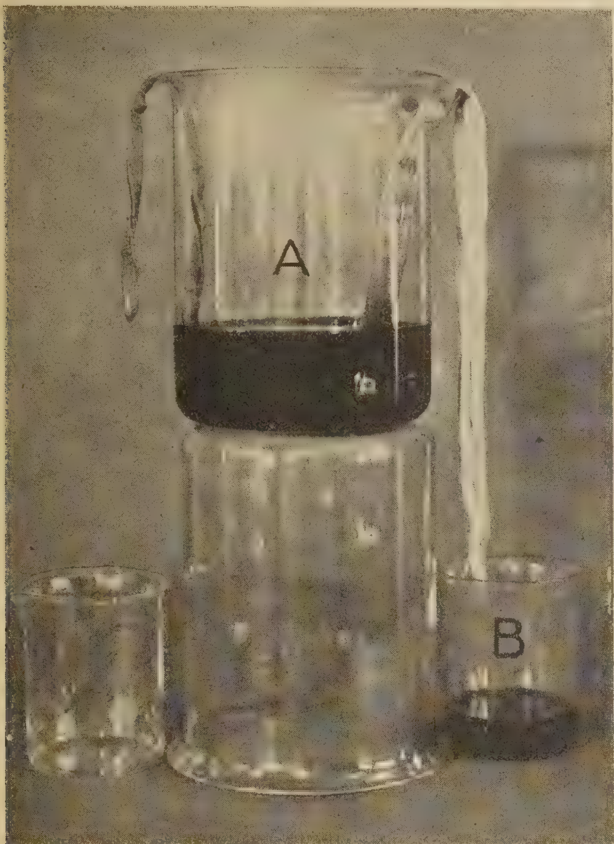


FIGURE 19.—ILLUSTRATION OF CAPILLARITY

below the surface of the liquid in the container, performs like a syphon and transfers the liquid from container A to container B.

When this photograph was taken the liquid was dropping into container B from the end of the cheesecloth wick at the rate of 1 drop every 38 seconds or

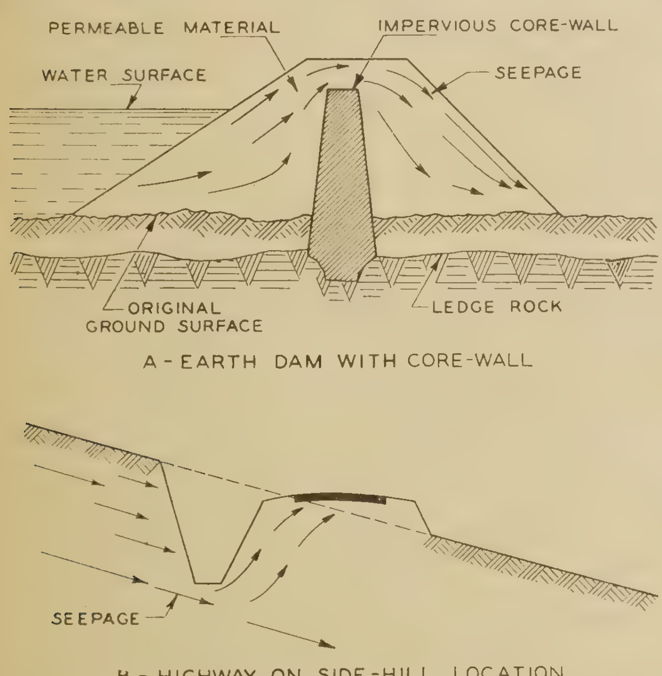


FIGURE 20.—ILLUSTRATIONS OF CAPILLARY ACTION IN SOILS

1 gram every 10 minutes. This equals 1 gallon every 631 hours. At times during the experiment the rate was as much as 1 gallon every 12 days.

This simple experiment explains the occurrence of seepage in the lower face of an earth dam, even though the top of the impervious corewall is higher than the adjacent water surface, Figure 20, A, and in subgrades in spite of ditches, as shown in Figure 20, B.

The maximum distance through which water may be forced by capillarity depends upon the surface tension of the water and the size of the soil pores, and increases as the size of the soil pores decreases, the temperature of the water remaining constant (*1*, *5*). The rate at which capillary moisture travels depends upon the capillary tension, upon the frictional resistance furnished by the walls of the pores to the flow of water, and upon the rate at which capillary equilibrium is destroyed by evaporation, the formation of ice crystals, or change in ground water elevation (*11*).

FIGURE 21.—SOIL IN FACE OF CUT ERODED BECAUSE OF ITS EXPANSIVE PROPERTIES

PROPERTIES AFFECTING EXPANSION AND SHRINKAGE OF SOILS ANALYZED

The extent to which water will be absorbed depends upon both the capillary properties and the degree of cohesion possessed by the soil. Water entering cohesionless soils through capillary action may cause the grains to separate to such an extent that the soil quickly disintegrates, as shown in Figure 21. A sufficient amount of cohesion existing between the soil particles will prevent the entrance of water in an amount sufficient to cause the soil to lose stability, unless the soil is manipulated. The amount of cohesion possessed by a soil of given constituents depends upon both the moisture content and the state of compaction of the soil. Therefore the relative amount to which the soil will expand depends upon both the degree of consolidation and the moisture content of the soil before wetting.

Each of 90 soil cakes were compressed in the wet state in the subgrade laboratory of the Bureau of

Public Roads, under a load of 3 kilograms per square centimeter. The load was then reduced to 0.028 kilogram per square centimeter and the cakes were permitted to absorb water. Subsequently two disks, each 1 square inch in area, were cut from each of the 90 soil cakes. One of these disks, in the wet state, was immersed in water and its counterpart was first allowed to dry to constant weight in the air and was then immersed in water.

Eight of the 90 disks, containing less than 12 per cent clay and immersed in the wet state disintegrated after being immersed for periods averaging 73 days. Of the 8 corresponding disks immersed in the dry state, 6 disintegrated after being immersed for an average period of 7 minutes and the remaining 2 swelled and cracked in appreciable amount but did not completely disintegrate after being immersed for a period of 25 days. Sixty-eight of the 90 disks containing, with several exceptions, clay 13 to 77 per cent and immersed in the wet state, remained intact after being immersed for an average period of nine months. Of the corresponding 68 disks immersed in the dry state, 26 disintegrated after being immersed for an average period of 10 minutes, 41 disintegrated after being immersed for an average period of 1 hour, and the remaining disk cracked and swelled in appreciable amount after being immersed for a period of 10 days.

As additional evidence that cohesion in soils tends to prevent their expansion and disintegration due to water absorption reference is made to Figure 22. The soil cakes shown in this figure were made up from soil

Figure 22.—Soil Samples Containing Water-Gas Tar in Varying Amounts, Photographed After Immersion in Water for a Period of Two Weeks

samples taken at different depths in a subgrade located at Arlington, Va., which was treated with water-gas tar in 1923. After their removal from the subgrade in 1929, these cakes were compressed in a semidry state, dried to constant weight in the air, and immersed in water for a period of two weeks. The two cakes shown on the left of Figure 22 contain tar in appreciable amount and exhibit no signs of disintegration. The third cake from the left contains but a small amount of tar and has crumbled slightly along the top edges. The fourth cake from the left contains but a slight trace of tar and shows crumbling in appreciable amount near the top. The two cakes on the right contain no visible trace of tar and have crumbled to a still greater extent.

Reduction in soil volume as illustrated in Figure 23 is caused by reduction in the moisture content of the soil. This action depends upon the capillary force exerted as the moisture content is reduced by evaporation and upon the resistance furnished by the soil particles to being consolidated. The theory of shrinkage in soils may be briefly stated as follows (*12*): Assume a soil with its pores completely filled with moisture to be in the liquid state. Under these conditions the contractive force exerted by the surface tension of the water is practically zero. As water evaporates from

Figure 23.—Large Shrinkage Cracks Over 3 Inches Wide and Over 1 Foot Deep

the sample the capillary tension exerts on the outer surface of the sample a uniformly distributed force acting at every point perpendicular to the outer surface of the sample, and tending to draw the particles in. As the soil sample becomes smaller and smaller its resistance to further shrinkage correspondingly in-

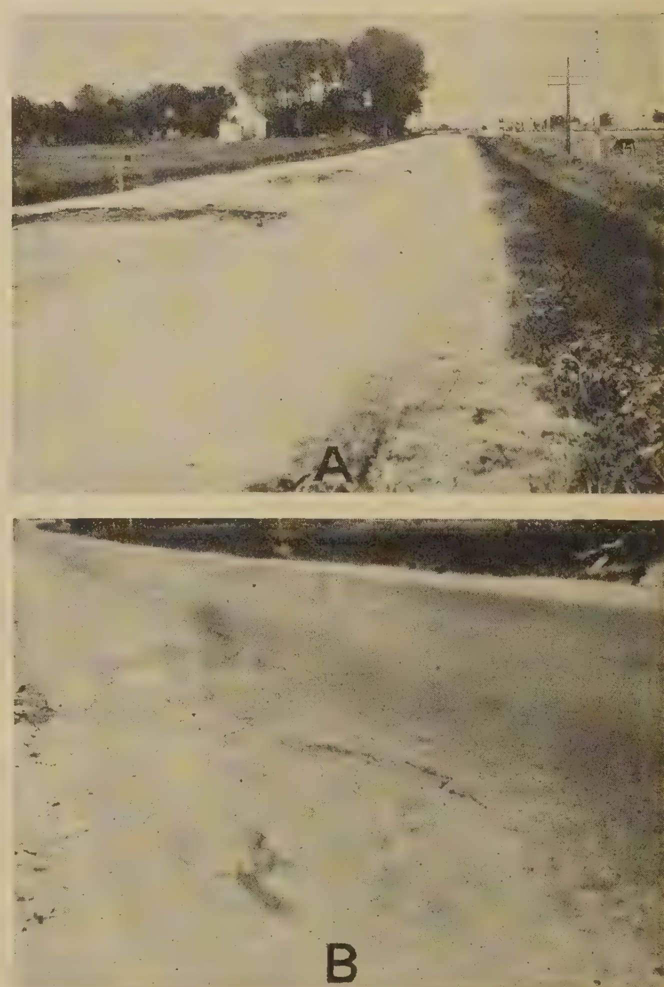
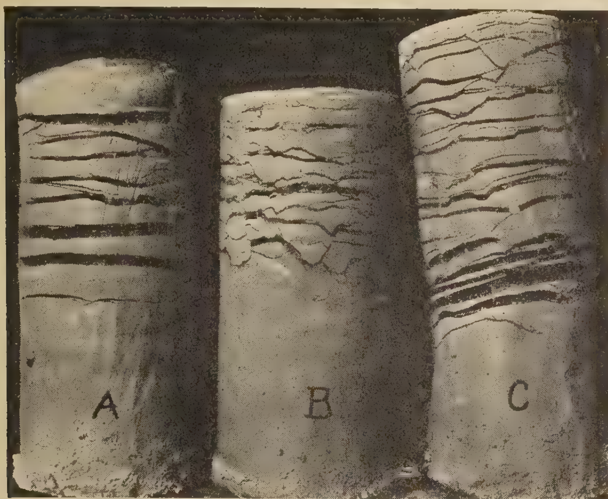


Figure 24.—Examples of Frost Damage in Roads. A.—Gravel Road Heaved by Frost Action. B.—Bituminous Macadam Road Damaged by Frost Action in Subgrade



Ice lenses in test cylinders frozen by Prof. Stephen Taber

Ice lenses observed by F. C. Lang in soil in Minnesota

FIGURE 25.—EXAMPLES OF ICE SEGREGATION IN SOILS

creases. Finally the soil attains a volume at which the resistance of the soil sample to further reduction in volume just equals the capillary pressure exerted by the evaporating moisture. Further evaporation will not appreciably decrease the volume of the soil sample. The moisture content of the soil at this state of equilibrium is termed "the shrinkage limit" and is discussed in Part II of this report (p. 28).

DETRIMENTAL HEAVING CAUSED BY SEGREGATION OF WATER WHEN FREEZING

Heaving of soils due to frost action (fig. 24, A and B) is caused by an increase in total moisture content occurring as ice layers or crystals (*13, 14, 15*). Its magnitude depends upon the rate of temperature change, the moisture content of the soil prior to freezing, the proximity of additional water and the rate at which capillary flow occurs (*13, 14, 15, 16*).

The formation of well defined ice layers in the soil (figs. 25,[4] and 26) depends on three physical phenomena—

(*a*) The tendency of water particles contained in soil pores of the larger capillary dimensions to freeze at either normal freezing or slightly less than normal freezing temperatures (−1°, −4° C.).

(*b*) The tendency of water particles contained in soil pores of the smaller capillary dimensions to resist freezing at abnormally low temperatures (as low as −70° C.).

(*c*) The tendency of water particles of freezable size, during the process of freezing, to draw to themselves from adjacent fine capillaries the small particles of water which individually do not freeze at ordinary freezing temperatures (*14*). When drawn to the existing ice crystal these small water particles freeze and increase in size. Continuation of this process causes the original ice crystals to increase in size as long as they are being supplied with small water particles drawn up through the fine capillaries from the ground water supply.

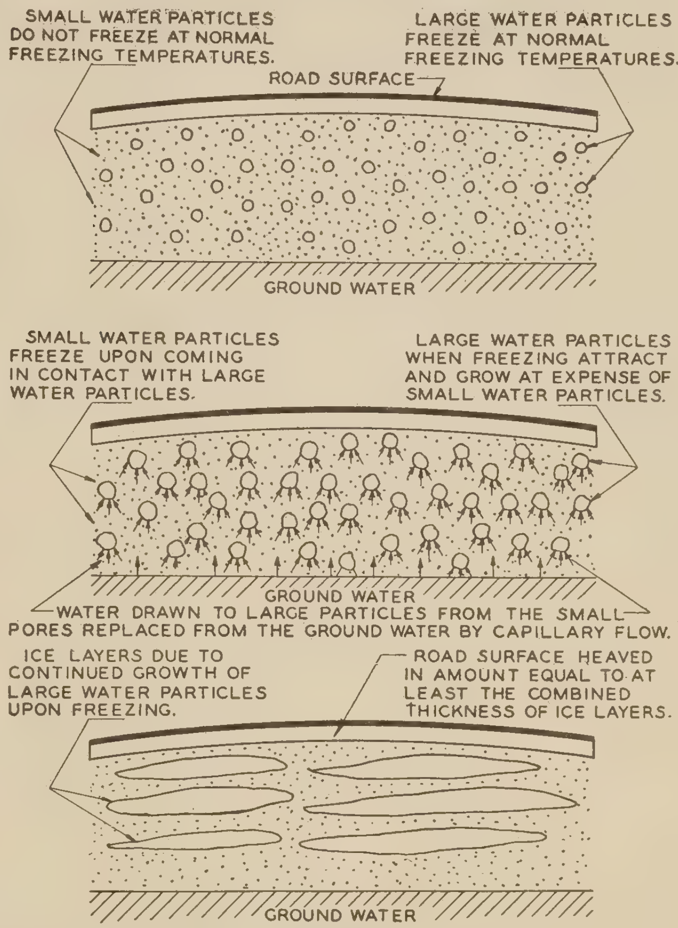


FIGURE 26.—DIAGRAM ILLUSTRATING THE PHYSICS OF FROST HEAVE

Whether or not frost heave will occur depends upon the quantity of moisture capable of being raised to a given height above the water table in a given time. Neither the height to which water will rise by capillarity nor the rate of such rise is alone the determining factor. Just as the amount of water furnished by a pipe depends upon the pressure acting on the water, the diameter of the pipe and the frictional resistance to flow, the raising of a quantity of water sufficient to produce frost heave in the subgrade at a given height above the ground water elevation (disregarding the rate at which capillary equilibrium is destroyed) depends upon the force of capillarity, the area of pore space, and the frictional resistance.

Capillary pressure varies inversely with the diameter of the pores. The frictional resistance to flow through a soil is a function of the surface area of the soil particles and consequently increases with a decrease in grain size at a much greater rate than does the capillary pressure. Therefore, in order to furnish capillary

[4] Reproduced from the article, Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements, by Stephen Taber, Public Roads, vol. 11, No. 6, August, 1930.

moisture in detrimental amounts the pore size must be small enough to furnish appreciable capillary pressure but large enough to prevent too much frictional resistance to flow.[5]

As an indication of the amounts of water furnished by capillarity, it has been estimated that certain Iowa silts (*9*) are capable of raising water by capillarity at the rate of about 5 feet per year. It is indicated by observations in both New Hampshire and Minnesota that in extreme cases the rate may be as high as 10 or 15 feet per year.

FIGURE 27.—FROZEN CYLINDER, HALF SAND AND HALF CLAY. MUCH SEGREGATED ICE IN CLAY BUT NOT IN SAND. FURNISHED BY TABER EXPERIMENTS

Generally it can be stated that in cohesionless soils possessing but little capillarity no important frost heaves occurs because practically all of the contained water freezes at normal freezing temperatures and small unfrozen water particles do not exist in amounts sufficient to cause the frozen particles to suffer appreciable growth. Soils possessing relatively high capillarity and relatively low cohesion are likely to heave in very appreciable amounts. The contrast between sand and clay in this respect is shown in Figure 27.[4]

Highly cohesive soils may possess very high capillarity but the resistance to water flow in these soils is very great. Consequently in dense cohesive soils with low ground water level and absence of lateral seepage, only limited amounts of water are available for ice segregation. Under these conditions the soil adjacent to the growing ice crystals, as illustrated in Figure 28, is likely to dry out and shrink because of the loss of moisture (*16*). The ground water elevation in clays, therefore, must be comparatively high in order that important frost heave may occur.

IMPORTANT SUBGRADE CHARACTERISTICS INDICATED BY THE PRESENCE OF CERTAIN SOIL CONSTITUENTS

In the foregoing discussion certain relations have been shown to exist between the five basic physical characteristics of a soil (internal friction, cohesion, compressibility, elasticity, and capillarity) and the important characteristics of subgrade performance, i. e., resistance to lateral flow, the property of compressing vertically under applied loads with or without rebound upon the removal of load, resistance to sliding in cuts and fills, shrinkage or expansion due to changes in moisture, and heaving under frost action.

Both the state in which the soil exists and the properties of the soil constituents exert an important influence upon the occurrence of these characteristics. Thus, for instance, tests performed by Terzaghi (*4*) disclose that the ultimate bearing value (yield point) of a soil in the undisturbed state may be over twice that of the same soil in the disturbed state. Likewise the compressibility may be different when soil is in the disturbed state from what it is in the undisturbed state (*17*).

It is well known that the faces of cuts in certain of the loess soils in the Middle West may stand vertically for years in the undisturbed state whereas these same soils in the disturbed state lose stability easily and flow in the presence of water.

It is also common knowledge that sands may be either highly stable or "quick" depending on whether water, as for instance that furnished by waves, flows downward or, as in the case of that furnished by springs, flows upward through them.

In spite of these facts the importance of tests performed on soils in the disturbed state becomes apparent when one considers (*a*) that the soil composing the subgrade generally, exists in at least a partially manipulated state and that composing the sands-clay and other low type road surfaces exists in a completely manipulated state, and (*b*) that the constants furnished by such tests serve to disclose the presence of those soils constituents which exert an important influence not only on the five basic physical characteristics of subgrades referred to above but also upon the state in which soils may occur in the soil profile. When only the mechanical analysis was used to identify soils, many soils indicated by this determination to be similar in character when disturbed, were observed to be radically different in character when undisturbed in the field. This difference in field behavior was attributed to difference in the environment under which the soil developed or existed. Now it is found that soils similar in character according to the mechanical analysis may differ widely in character according to the physical tests and furthermore that many differences in field behavior of soils are due to differences in soil constituents.

FIGURE 28.—TYPE OF SHRINKAGE CRACKING LIKELY TO OCCUR WHEN WATER IS DRAWN OUT OF SOILS DUE TO FREEZING IN ADJACENT AREAS

The subgrade investigations have made it increasingly evident that the states in which soils are likely to exist under specified conditions, as well as the physical characteristics of the soils in different states, may be largely dependent upon the presence of certain soil constituents.

Among the many natural constituents of soils, a comparatively few serve to illustrate the properties which

[4] Reproduced from the article, Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements, by Stephen Tabor, Public Roads, vol. 11, No. 6, August, 1930.

[5] In 1909 Atterberg determined experimentally that the maximum height of capillary rise in 24 hours occurs in soil consisting primarily of particles 0.02 millimeter in diameter. This result was confirmed theoretically by Terzaghi (Erdbaumechanik, 1925, fig. 23).

may exert an important influence upon subgrade performance. These representative soil constituents are:

Gravel.—Particles larger than 2.0 millimeters in diameter (No. 10 sieve).

Coarse sand.—Particles between 0.25 (No. 60 sieve) and 2.0 millimeters in diameter.

Fine sand.—Particles between 0.05 and 0.25 millimeter in diameter.

Silt.—Particles between 0.005 and 0.05 millimeter in diameter.

Cohesive clay.—Particles smaller than 0.005 millimeter in diameter.

Gluey colloids.—According to Albert Atterberg (*18*), soil particles 0.002 millimeter or smaller in diameter show pronounced Brownian movements when suspended in water. This phenomenon, as explained by Oscar Edward Meinzer, indicates that the colloidal stage of fineness is reached at this size and, according to the kinetic theory of heat, the particles are so small that they are bounced about by the rapidly moving molecules with which they collide. The colloidal fraction reported in the mechanical analysis, however, consists of particles 0.001 millimeter and smaller in size.

Mica flakes.

Diatoms.

Peat.

Chemical constituents.—Certain chemicals, such as lime and magnesium, have a tendency to flocculate [6] fine-grained soils; others, such as sodium and potassium have a tendency to deflocculate or disperse fine-grained soils. (Fig. 29.)

Of these constituents, certain of which are shown in the photomicrographs of Figure 30, the gravel and sand are indicative of high internal friction, the silt, peat, and diatoms of detrimental capillarity; the clay and colloidal glues are indicative of cohesion and, together with the silt and when not flocculated, of compressibility; and the mica flakes, peat, and flocculated soils are indicative of elasticity.

Generally gravel and coarse sand furnish the main hardness and supporting strength of graded soil roads, especially in wet weather. Fine sand adds an embedment support to the coarse sand, and silt with low moisture content adds embedment for the sand. Clay and colloidal glues furnish cohesive and adhesive bond variable with their moisture contents (*11*).

The curves of Figure 31 show the great range of compressibility and expansion possessed by different soil constituents.

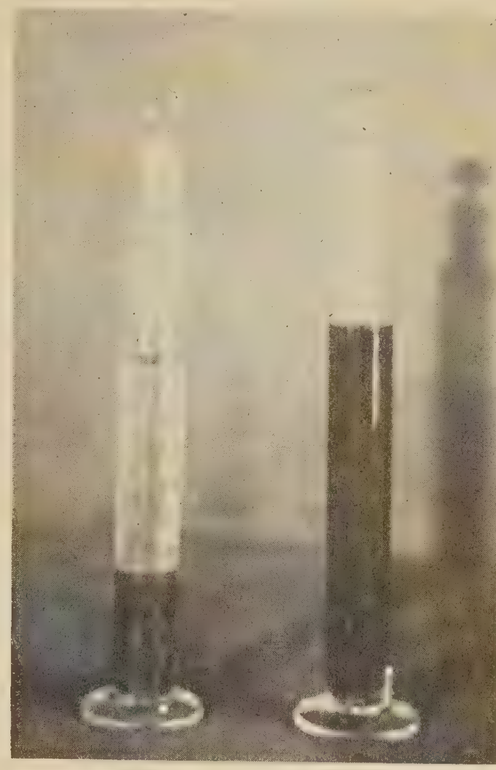


FIGURE 29.—SOIL IN FLOCCULATED AND DISPERSED STATE AFTER 24 HOURS SEDIMENTATION. FLOCCULATED SOIL (LEFT) SETTLES OUT RAPIDLY LEAVING CLEAR LIQUID ABOVE. DISPERSED COLLOIDS REMAIN IN SUSPENSION CAUSING LIQUID TO BE CLOUDY

SUBGRADES TENTATIVELY ARRANGED IN GROUPS

Because of the fact that the presence of certain soil constituents indicates the important soil properties, the subgrades may be arranged in groups representative of both soil constituents and characteristics. On this basis, the various soils have been tentatively arranged in groups with respect to their performance when used as subgrades, as follows (*7*):

UNIFORM SUBGRADES

Group A–1.—Well-graded material, coarse and fine, excellent binder. Highly stable under wheel loads, irrespective of moisture conditions. Functions satisfactorily when surface treated or when used as a base for relatively thin wearing courses.

Group A–2.—Coarse and fine materials, improper grading or inferior binder. Highly stable when fairly dry. Likely to soften at high water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from the top layer, or to become loose and dusty in long-continued dry weather.

Group A–3.—Coarse material only, no binder. Lacks stability under wheel loads but is unaffected by moisture conditions. Not likely to heave because of frost nor to shrink or expand in appreciable amount. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements.

Group A–4.—Silt soils without coarse material, and with no appreciable amount of sticky colloidal clay. Has a tendency to absorb water very readily in quantities sufficient to cause rapid loss of stability even when not manipulated. When dry or damp, presents a firm riding surface which rebounds but very little upon the removal of load. Likely to cause cracking in rigid pavements as a result of frost heaving, and failure in flexible pavements because of low supporting value.

Group A–5.—Similar to Group A–4, but furnishes highly elastic supporting surfaces with appreciable rebound upon removal of load even when dry. Elastic properties interfere with proper compaction of macadams during construction and with retention of good bond afterwards.

Group A–6.—Clay soils without coarse material. In stiff or soft plastic state absorb additional water only if manipulated. May then change to liquid state and work up into the interstices of macadams or cause failure due to sliding in high fills. Furnish firm support essential in properly compacting macadams only at stiff consistency. Deformations occur slowly and removal of load causes very little rebound. Shrinkage properties combined with alternate wetting and drying under field conditions are likely to cause cracking in rigid pavements.

Group A–7.—Similar to Group A–6, but at certain moisture contents deforms quickly under load and rebounds appreciably upon removal of load, as do subgrades of Group A–5. Alternate wetting and drying under field conditions leads to even more detrimental volume changes than in Group A–6 subgrades. May cause concrete pavements to crack before setting and to crack and fault afterwards. May contain lime or associated chemicals productive of flocculation in soils.

Group A–8.—Very soft peat and muck incapable of supporting a road surface without being previously compacted.

NONUNIFORM SUBGRADES

Soils of these groups cause concrete pavements to crack or fault excessively and flexible types to fail or to develop rough riding surfaces.

Group B–1.—Nonuniform natural ground due to abrupt variation in soil characteristics or soil profile, or to frequent change in field conditions.

Group B–2.—Nonuniform subgrade due to nonuniform composition of fill.

Group B–3.—Nonuniform subgrade consisting in part of natural ground and in part of fill materials.

[6] Arrangement of a number of soil colloids into groups having approximately the size of silt particles.

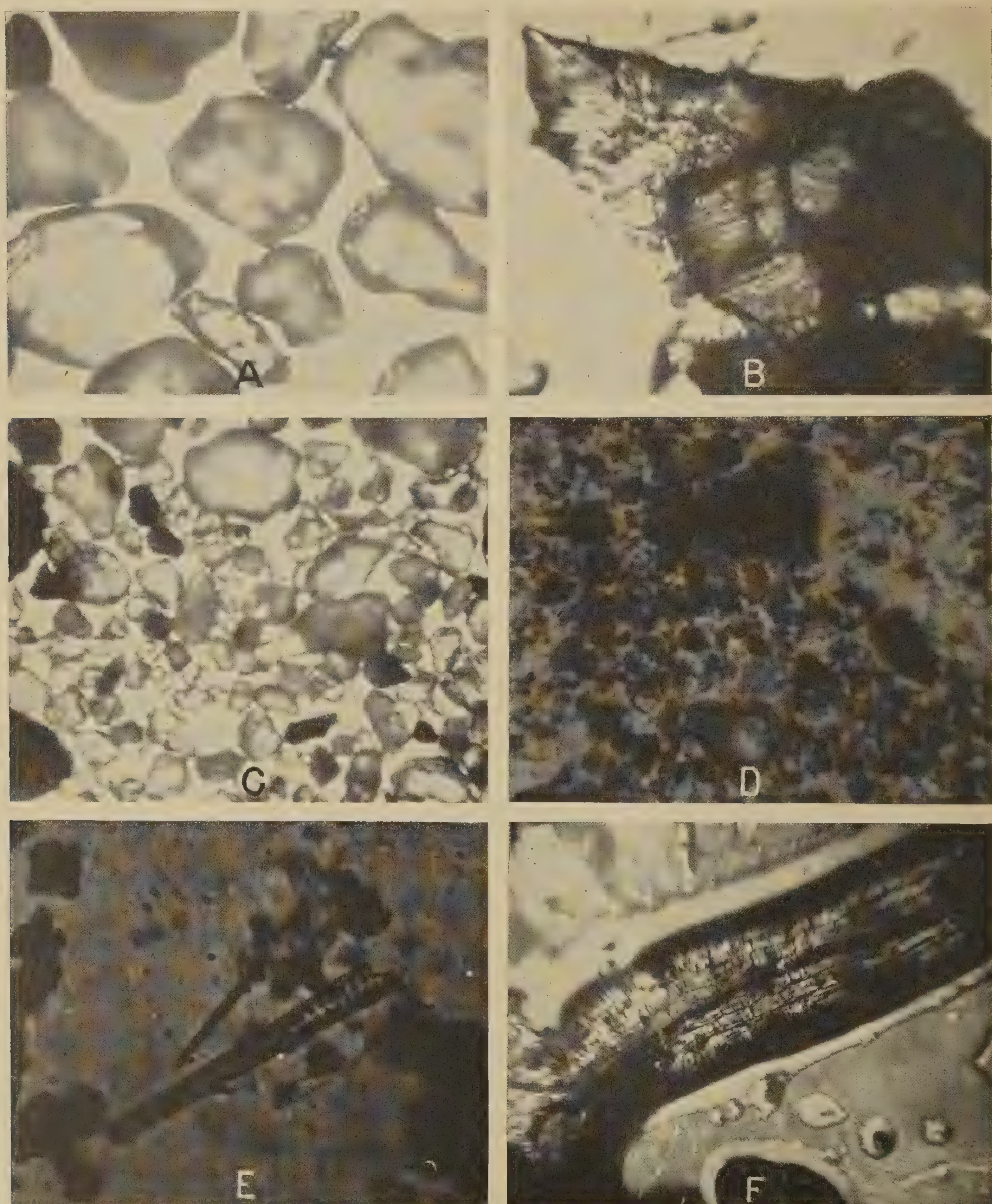


Figure 30.—Photomicrographs of Soil Constituents. A.—Beach Sand. B.—Angular Sand Grain. C.—Glacial Sand. D.—Soil Containing Diatoms. Note Spongelike Appearance. E.—Single Diatoms. F.—Peat-Bog Material. Note Fibrous Structure and Film of Water Surrounding Individual Particles

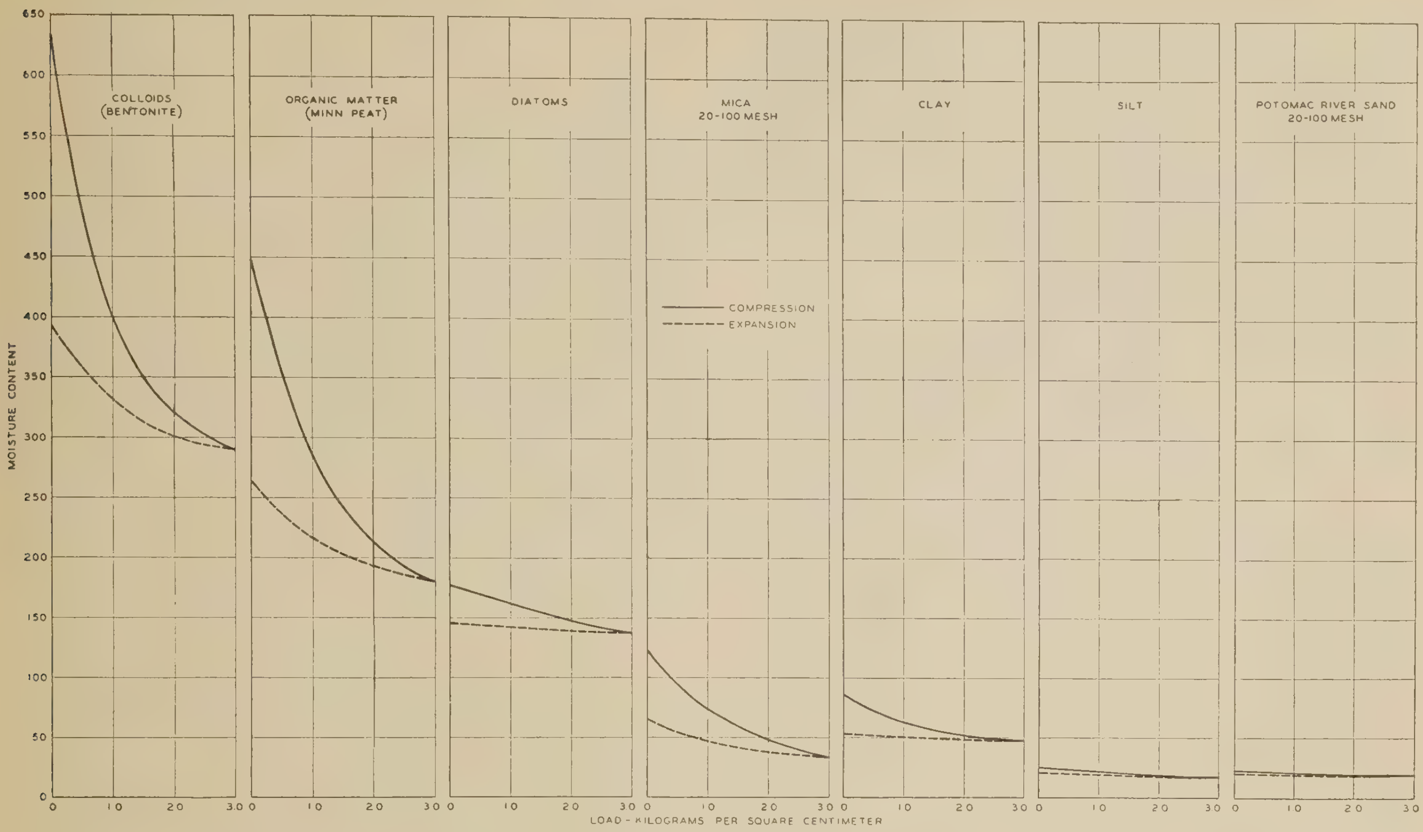


FIGURE 31.—COMPRESSION AND EXPANSION CURVES FOR REPRESENTATIVE SOIL CONSTITUENTS

The foregoing is not intended to be a rigid and final soil classification. It does, however, arrange the uniform soils according to those physical characteristics which are important with respect to subgrade performance and in this manner constitutes the basis for a final classification.

The terms "clay," "silt," and "sand" are used in defining the different groups. These terms, however, refer to the physical properties generally assumed to be possessed by these constituents rather than to definite grain sizes. Thus in the identification of members of the different soil groups grain size is subordinate to physical properties.

The advantage furnished by this method of identification is illustrated as follows: Assume for instance a bulky grained material, Figure 32, A and B, passing the No. 20 and retained on the No. 100 sieve. According to grain size it is sand. Physically it furnishes excellent firm support for a road surface. It does not possess either detrimental elasticity or capillarity and is, consequently, a Group A–3 subgrade.

A similar material containing an appreciable amount of micaceous particles, Figure 32 C, passing the No. 20 and retained on the No. 100 sieve would also be sand, according to grain size. Physically, however, this material possesses both elasticity and capillarity in detrimental amounts; and, since it has no cohesion, it is a Group A–5 subgrade.

In this grouping, therefore, sand instead of being a material characterized only by a specific grain size becomes a Group A–3 subgrade having internal friction, no cohesion, and capillarity in amount insufficient to cause detrimental expansion. Members of the A–3 group are identified with respect to the ability possessed by their grains to resist sliding over each other.

Silt instead of being a material characterized only by a given grain size is divided into two groups: A–4 subgrade, which possesses internal friction, capillarity in appreciable amount, and neither cohesion nor elasticity in appreciable amount; and A–5, subgrade which has internal friction, both capillarity and elasticity in appreciable amount, and no cohesion.

Likewise, the clays are divided into two groups: A–6 subgrade which possesses both cohesion and capillarity in appreciable amount, but neither internal friction nor elasticity; and A–7 subgrade which possesses cohesion, capillarity, and elasticity in appreciable amount, but no internal friction.

This grouping with respect to both the physical properties of the soil and the soil constituents is illustrated diagrammatically in Figure 33. Different positions on the diagram represent different combinations of the five basic physical soil properties. Thus, for instance, a point plotted in the center of the diagram represents the perfect A–3 subgrade, having a maximum amount of internal friction. As the position of the point shifts from the center to the outer boundary of the diagram the magnitude of internal friction decreases gradually from a maximum to negligible amounts. As a point shifts along the circumference from the bottom to the top of the chart the indicated soil changes gradually from the compressible to the elastic type.

Adding compressible materials in increasing amounts therefore gradually changes a group A–3 sand first to a nonplastic variety of the A–2 subgrade; second, to a well graded A–1 subgrade; third, to a plastic variety of the A–2 subgrade and last to either a Group A–4 or Group A–6 subgrade.

The friable variety (sand predominating) differs from the plastic variety (clay predominating) of the Group A–2 subgrade in the following manner. A friable variety, to remain stable, requires the cohesion furnished by capillary pressure and therefore is likely to be highly stable on damp and unstable on thoroughly

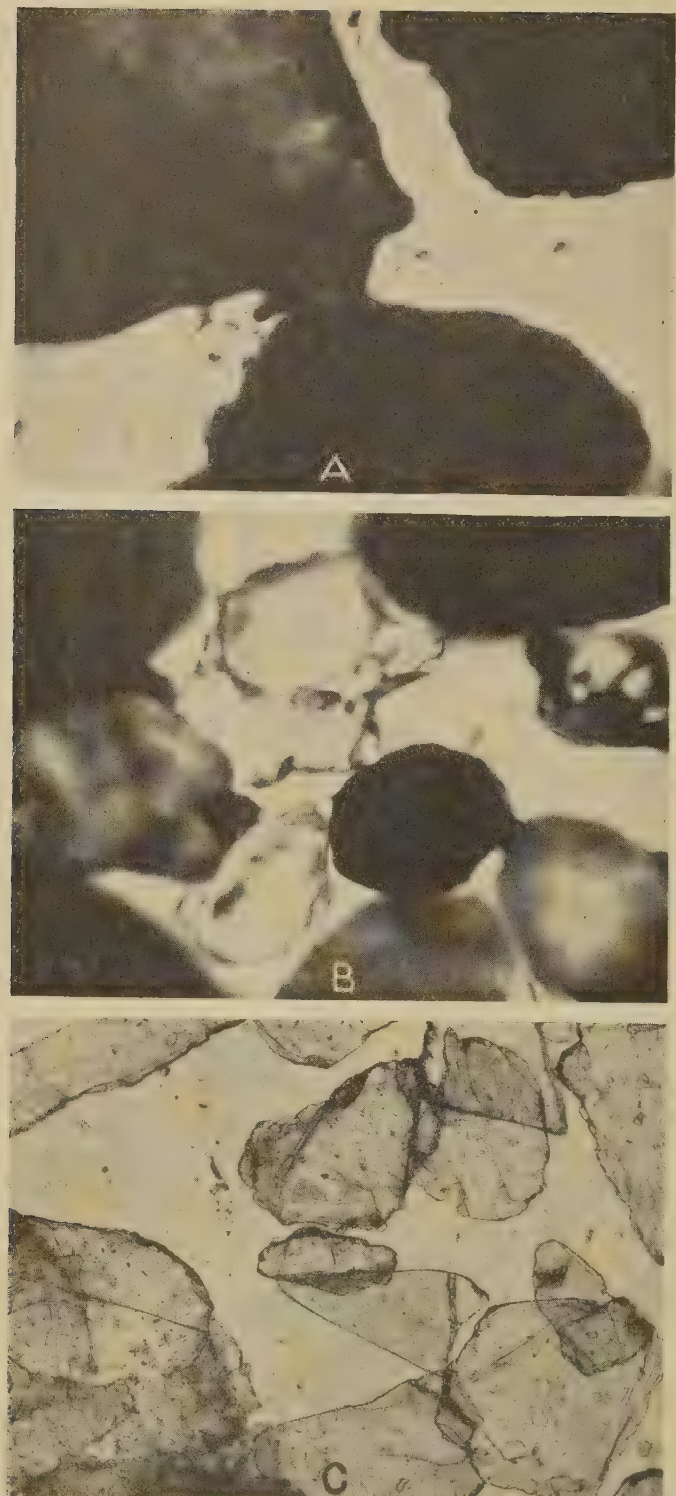


FIGURE 32.—PHOTOMICROGRAPHS OF MATERIAL PASSING THE No. 20 AND RETAINED ON THE No. 100 SIEVE. A.—CRUSHED ANGULAR SAND. B.—SUBANGULAR RIVER SAND. C.—MICA FLAKES

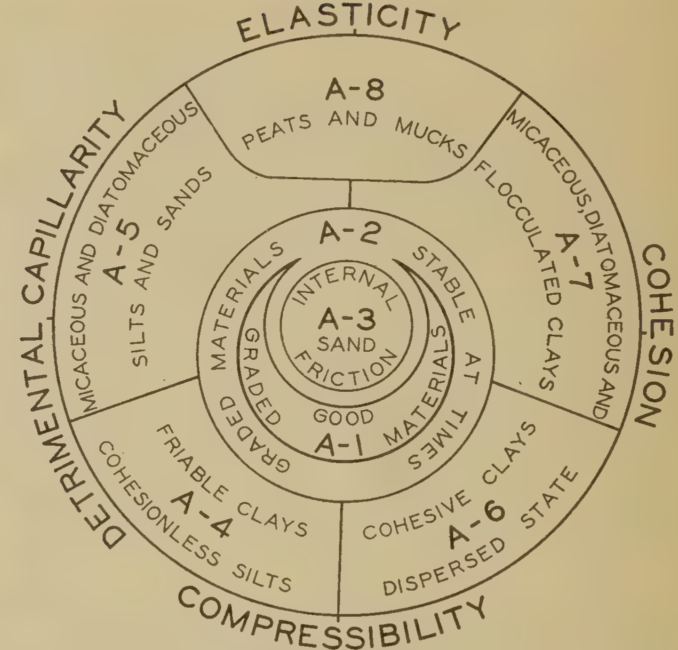


FIGURE 33.—DIAGRAM ILLUSTRATING CHARACTERISTICS OF UNIFORM SUBGRADE GROUPS

dry subgrades. The plastic variety, in contrast, remains stable when fairly dry and is apt to soften on damp subgrades.

Thus one sees that the uniform subgrade groups may be defined with respect to properties summarized as follows:

Group A–1.—High internal friction, high cohesion, no detrimental shrinkage, expansion, capillarity, or elasticity.

Group A–2.—High internal friction and high cohesion only under certain conditions. May have detrimental shrinkage, expansion, capillarity, or elasticity.

Group A–3.—High internal friction, no cohesion, no detrimental capillarity, or elasticity.

Group A–4.—Internal friction variable, no appreciable cohesion, no elasticity, capillarity important.

Group A–5.—Similar to A–4 and in addition possesses elasticity in appreciable amount.

Group A–6.—Low internal friction, cohesion high under low moisture content, no elasticity, likely to expand and shrink in detrimental amount.

Group A–7.—Similar to A–6 but possesses elasticity also.

Group A–8.—Low internal friction, low cohesion, apt to possess capillarity and elasticity in detrimental amount.

PART II: A DISCUSSION OF THE SOIL CONSTANTS AND THE SOIL IDENTIFICATION CHART[a]

THE FIRST part of this series of articles, published in the June, 1931, issue of PUBLIC ROADS (pp. 1 to 20), dealt with the five major physical properties of soils, internal friction, cohesion, compressibility, elasticity, and capillarity, and their relation to subgrade performance. It was shown that the presence of these properties in varying degrees depends very largely on the soil constituents, although the state in which a soil exists is a factor which can not be neglected. A method was outlined for classifying soils according to the predominance of certain constituents in them, and therefore, in a general way, according to their physical characteristics. The present report (Part II) discusses the test constants which serve to identify the constituents of soils and their resulting properties; and the soil identification chart, which is a convenient means of analyzing the data given by the laboratory tests. Part III describes the manner in which the soil identification chart is used in actual practice.

MECHANICAL ANALYSIS LIMITED AS AN INDICATOR OF SUBGRADE EFFICIENCY

From the information given in Part I it is clear that a knowledge of the size of soil particles does not furnish a complete identification of subgrade characteristics. Indeed, the mechanical analysis is very much limited in its ability to disclose accurately the size of the smaller soil particles, which must be determined by means of some method of sedimentation instead of by sieves. As a consequence, for grains smaller than about 0.074 millimeter (No. 200 sieve) in diameter, the grain diameters given by the mechanical analysis instead of being the diameters of the grains tested are the diameters of spheres which, according to Stokes law, (*20*), settle in water at a rate equal to that of the particles of soil being analyzed. A further inaccuracy is introduced by the fact that the rate at which small soil particles settle depends upon the extent to which the soil is dispersed; and this in turn depends upon both the method of agitation and the chemical reagent used in dispersing the soil.

Thus, for instance, a soil which, according to the mechanical analysis, contains 50 per cent of clay, does not necessarily contain 50 per cent of particles having diameters smaller than 0.005 millimeters. The correct interpretation of the test is that 50 per cent of the particles settle through water at a rate equal to that of spherical particles not exceeding 0.005 millimeters in diameter.

Figure 34,[7] which shows photomicrographs of soil suspensions at different periods of time after dispersion, discloses how very much the shape of soil particles is likely to vary from the spherical.

Table 4, furnished by L. B. Olmstead, L. F. Alexander, and H. E. Middleton (*20*), illustrates how the type of dispersing agent influences the silt and the clay contents furnished by the mechanical analysis. As shown in this table, the value obtained for Houston black clay may be 45 per cent or 64 per cent, depending on whether sodium hydroxide or sodium oxalate is used as a dispersing agent. Furthermore, the clay fraction depends upon the degree of agitation used during dispersion. Too little fails to separate the particles sufficiently, causing the indicated clay content to be too small. Too much agitation not only separates the particles but may also break them into smaller pieces, thus causing the indicated clay content to be too large.

TABLE 4.—*Yield of clay obtained by use of various dispersing agents* [1]

Soil type and source	Ammonia		Sodium hydroxide		Sodium carbonate		Sodium oxalate	
	Percentage of clay particles obtained							
	0.005 mm.	0.002 mm.	0.005 mm.	0.002 mm.	0.005 mm.	0.002 mm.	0.005 mm.	0.002 mm.
Wabash silt loam (Nebraska)	30.7	24.1	33.2	28.5	33.1	28.3	34.5	29.6
Houston black clay (Texas)	38.4	26.4	44.5	30.6	61.3	53.5	63.8	53.8
Carrington loam (Iowa)	7.8	6.4	24.9	21.8	23.4	19.9	24.7	22.9

[1] Average of duplicate determinations.

It should be remembered also that when grain size is computed from a knowledge of rate of settlement of particles in water, it is assumed that all of the grains in the soil mass being tested possess equal specific gravities, and this is not necessarily true.

The fact that mechanical analysis is inadequate to identify the characteristics of subgrade soils is illustrated by the gradings typical of the uniform subgrade groups, which are given in the following paragraphs. The two terms, "effective size" and "uniformity coefficient," used in this discussion, are defined as follows:

The effective size is the maximum size of the smallest 10 per cent, by weight, of the soil particles. On a soil accumulation curve (fig. 35) the value of the effective size is given by the abscissa of the point on the curve having the ordinate 10 per cent.

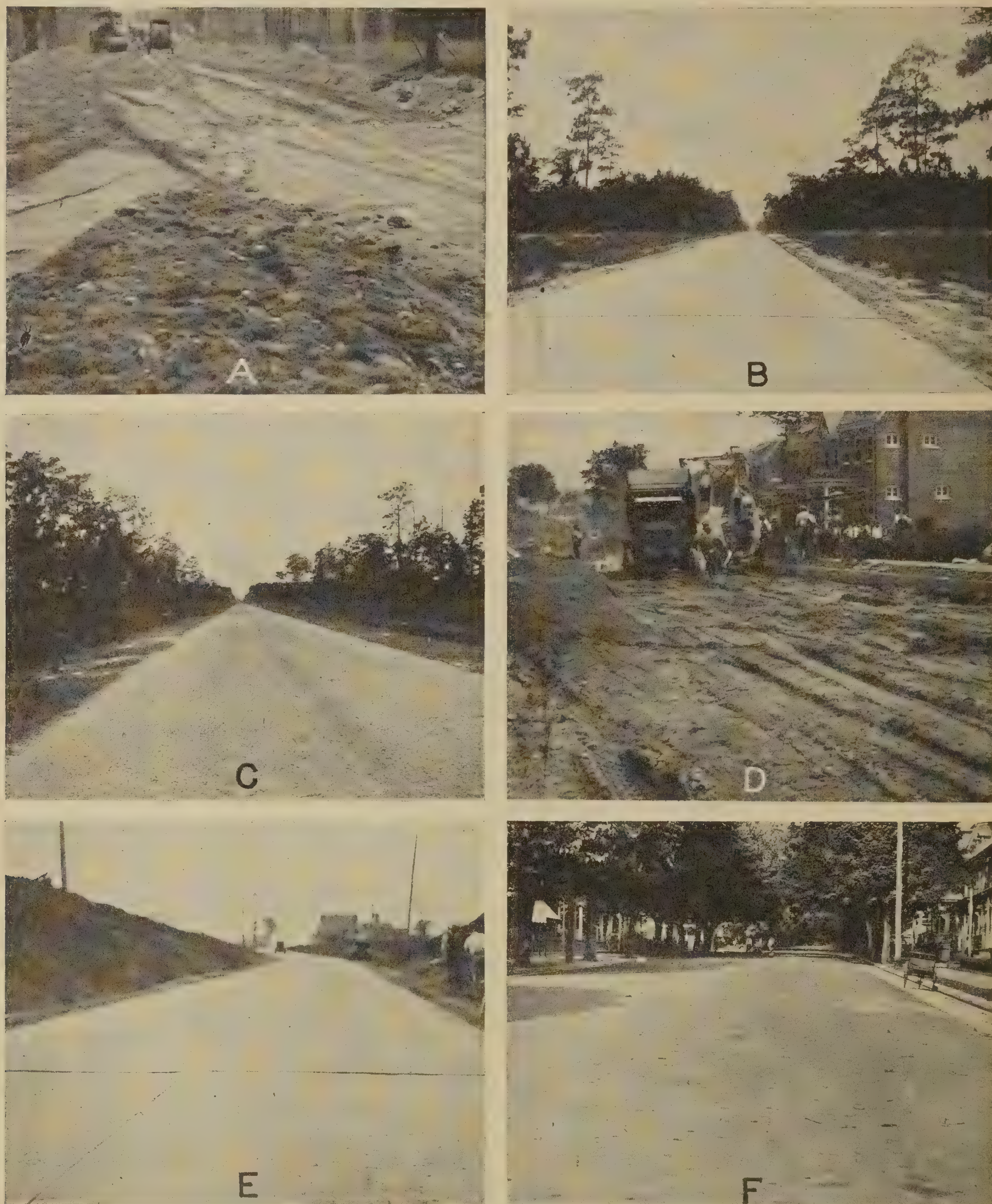
The uniformity coefficient is the ratio between the maximum size of the smallest 60 per cent, by weight, of the soil particles, and the effective size. Its value may be obtained from a soil accumulation curve by computing the ratio between the abscissa of a point whose ordinate is 60 per cent and the abscissa of a point whose ordinate is 10 per cent.

The typical gradings are as follows:

Group A-1 subgrade.—Material retained on the No. 10 sieve, not more than 50 per cent. The soil mortar, that fraction passing the No. 10 sieve, composed as follows: Clay, 5 to 10 per cent; silt, 10 to 20 per cent; total sand, 70 to 85 per cent; and coarse sand (retained on the No. 60 sieve) 45 to 60 per cent. Effective size approximately 0.01 millimeter and uniformity coefficient greater than 15.

The soil accumulation curve of Figure 35 shows graphically the average grading of stable soil mortars. In this case the

[a] Reprinted from PUBLIC ROADS, vol 12, No. 5, July, 1931.
[7] The numbers of figures, tables, footnotes, and equations are consecutive with those of Part I of this report (Public Roads, June, 1931). Italic numerals in parentheses refer to the bibliography given on page 49.



EXAMPLES OF SUBGRADE CLASSIFICATION AND PAVEMENTS LAID ON DIFFERENT TYPES OF SUBGRADE. A, BASE COURSE BEING CONSTRUCTED OF GROUP A–3 MATERIAL; B, CONCRETE PAVEMENT LAID ON GROUP A–3 SUBGRADE IN FLORIDA; C, BITUMINOUS MACADAM WEARING SURFACE ON GROUP A–4 BASE. D, GROUP A–5 SUBGRADE SHOWING REBOUND AFTER BEING ROLLED. E, EXCELLENT CONCRETE PAVEMENT CONSTRUCTED ON GROUP A–5 SUBGRADE. F, SURFACE TREATED MACADAM CONSTRUCTED ON AN EXCELLENT A–6 SUBGRADE.

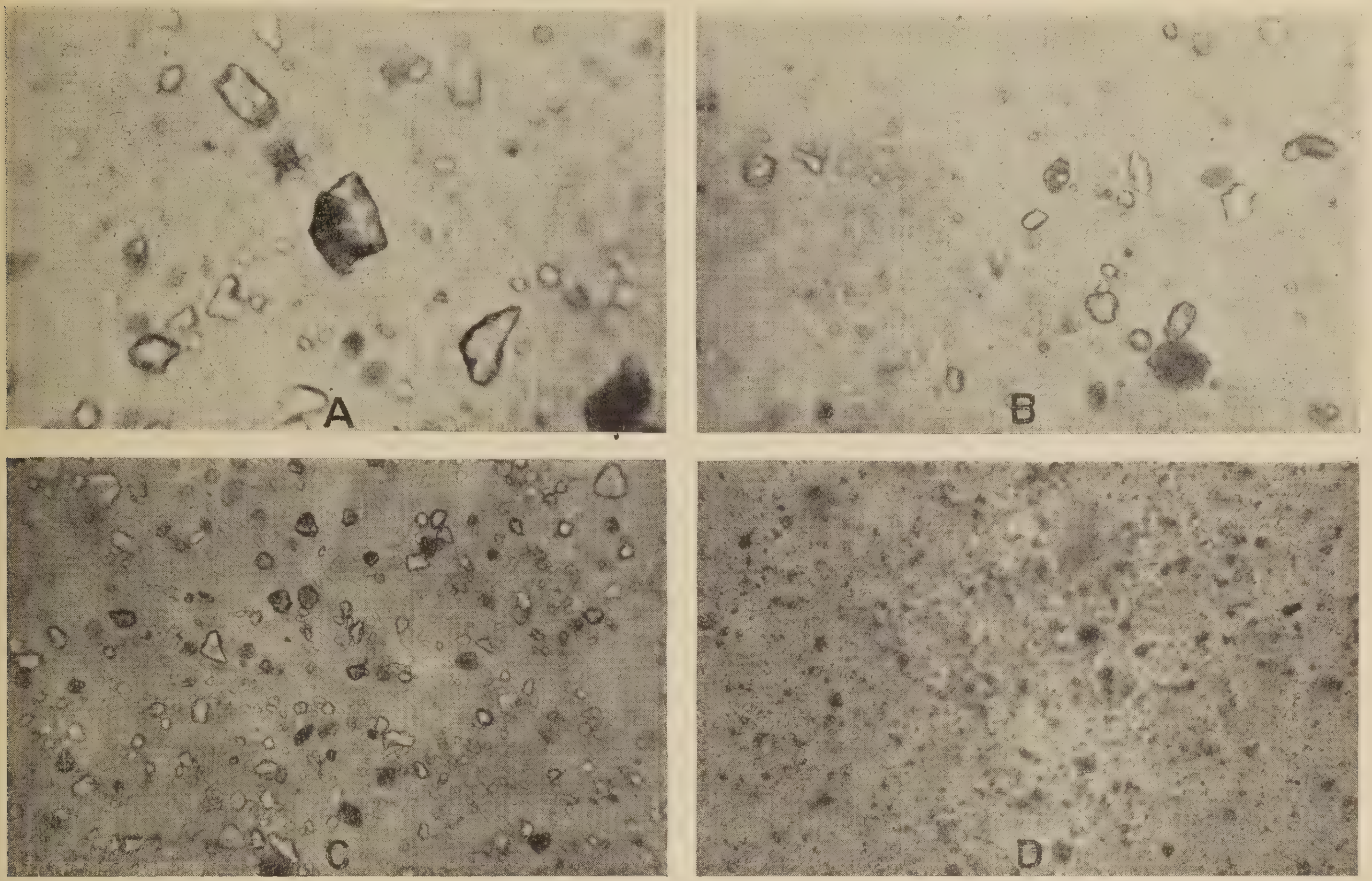


FIGURE 34.—PHOTOMICROGRAPHS OF SOIL SUSPENSION AT DIFFERENT PERIODS OF TIME AFTER DISPERSION: A, AFTER 1 MINUTE; B, AFTER 2 MINUTES; C, AFTER 5 MINUTES; D, AFTER 15 MINUTES. NOTE REDUCTION IN SIZE OF PARTICLES IN SUSPENSION AS TIME OF SEDIMENTATION INCREASES

effective size is 0.01 millimeter and the uniformity coefficient is $\frac{0.43}{0.01}$, or 43.

Group A–2 subgrade.—Not less than about 55 per cent of sand in the soil mortar.

Group A–3 subgrade.—Effective size not likely to be less than 0.10 millimeter.

Group A–4 subgrade.—Likely to contain sand in amount less than 55 per cent.

Group A–5 subgrade.—Same as group A–4.

Group A–6 subgrade.—Likely to contain more than 30 per cent clay.

Group A–7 subgrade.—Same as group A–6.

Group A–8 subgrade.—Grading not significant.

From the facts given in the preceding discussion it is plain that mechanical analysis gives only approximately the diameter of soil particles of small size, and that the size of grain, even if accurately known, is a very imperfect criterion of subgrade soil characteristics. In order to identify these characteristics it is necessary to employ constants which disclose the degree to which particular physical properties are present in a given soil.

SIGNIFICANCE OF TEST CONSTANTS DISCUSSED

Among the constants which have been suggested as aids in identifying the important subgrade properties are the liquid limit, the plastic limit, the plasticity index, the shrinkage limit, the centrifuge moisture equivalent, the field moisture equivalent, the shrinkage ratio, the volumetric change, and the lineal shrinkage. In Table 5 are listed values of these test constants for a group of representative subgrade soil constituents. In the tests from which the values given in Table 5 were derived the following materials were used:

Sand.—Potomac River sand; that fraction passing the No. 20 and retained on the No. 100 sieve.

Silt.—Silty sand soil obtained in Rock Creek Park, District of Columbia.

Clay.—Yaguajay clay (clay about 70 per cent and silt about 30 per cent) from Cuba. Furnished by H. H. Bennett, United States Bureau of Chemistry and Soils.

Colloids.—Bentonite. According to C. S. Ross and C. V. Shannon (*19*), out of five clay minerals contained in bentonite, three are micaceous, one is platy crystaline, and one is amorphous. According to the hydrometer analysis 99 per cent of the bentonite particles are smaller in diameter than 0.05 millimeter (silt), 85 per cent are smaller than 0.005 millimeter (clay), 80 per cent are smaller than 0.001 millimeter (colloids), and 79 per cent are smaller than 0.0005 millimeter.

Mica flakes.—That fraction passing the No. 20 and retained on the No. 100 sieve.

Diatoms.—Celite: 95 per cent of particles smaller in diameter than 0.05 millimeter and 61 per cent smaller than 0.005 millimeter.

Peat.—Everglade peat, Florida, furnished by the United States Bureau of Chemistry and Soils. Sixty-five per cent of particles smaller in diameter than 0.05 millimeter and 18 per cent of particles smaller than 0.005 millimeter.

In addition to the tests referred to above both a compression and a slaking test may assist in identifying those binder clays of the group A–1 and A–2 subgrades which, because of certain chemical constituents, are inclined to "set up" upon drying. These same tests performed upon soil samples both treated and untreated with bituminous materials serve to disclose to some degree the increase in stability furnished by the treatment. Chemical tests may also be required to disclose those soil chemicals which are likely

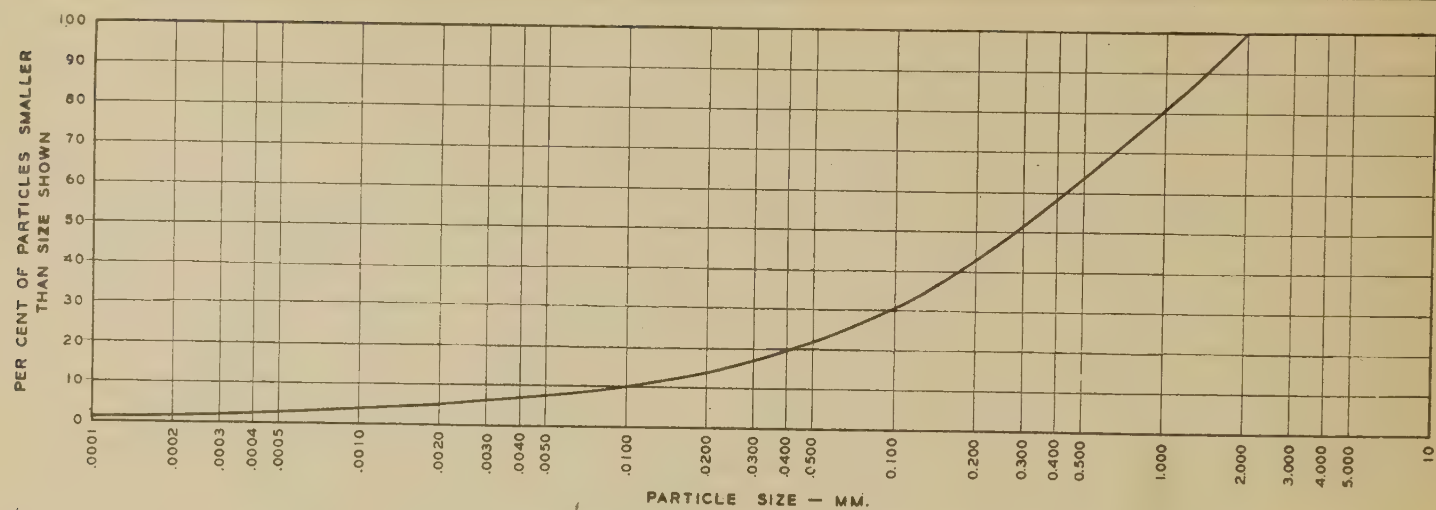


FIGURE 35.—GRAIN SIZE ACCUMULATION CURVE FOR GOOD BINDER

TABLE 5.—*Laboratory test results given by representative soil constituents*

Soil constituent	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent		Volumetric change	Lineal shrinkage
			Limit	Ratio	Centrifuge	Field		
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*	*Per cent*	*Per cent*
Sand	20	[1] 0			4	25	0	0
Silt	27	7	19	1.8	22	22	5	2
Clay	100	54	11	2.1	70	55	92	19
Colloids	399	354	6	2.0	(2)	86	160	27
Colloids, 50 per cent	174	154	12	1.9	291	54		
Mica	123	0	160	0.52	159	142	[3] −9	[3] −3
Diatoms	163	0	134	0.5	221	212	39	10
Peat	136	0	44	0.9	90	121	69	16

[1] Indicates nonplastic soils without plastic limits.
[2] Centrifuge moisture equivalent could not be determined because of very great expansion suffered by pure bentonite when being saturated.
[3] Negative value indicates expansion of mica on drying, discussed subsequently.

to exert a detrimental influence upon concrete pavements and structures. The compression, slaking, and chemical tests are not discussed in this report.

In order to use the constants intelligently one must thoroughly understand their physical significance. The soil identification chart, discussed subsequently in this report, assists in readily identifying the characteristics of many soils. The interpretation of results furnished by tests performed on many other soils, however, can be accomplished only by an intimate knowledge of physical phenomena occurring when the soils are subjected to test. For this reason an attempt is made to explain these physical phenomena in detail.

In this connection, it should be noted that the significance of a constant may vary, depending upon the degree of capillarity possessed by the soil. For this reason soils are referred to as either expansive or nonexpansive soils, according to their degree of capillarity.

The expansive soils, silt, clay, etc., are those whose capillarity is sufficient to cause swelling, shrinkage, or detrimental frost heave in appreciable amount. The nonexpansive soils are those generally termed sands, whose capillarity is not sufficient to produce these phenomena.

It is important to remember also that when a nonexpansive soil is being added to an expansive soil in increasing amounts the change in certain properties from the expansive to the nonexpansive variety may be abrupt instead of gradual. As will be shown later, sand added to clay in increasing amounts causes the resulting mixture to change its shrinkage characteristics very slowly until, when the percentage of sand becomes equal to a certain amount, the shrinkage characteristics of the mixture are abruptly changed from those of the clay to those of the sand. Important decrease in supporting value caused by increasing the moisture content of soils occurs not gradually but abruptly, when the moisture content of the soil exceeds a certain amount. In the same abrupt manner the state of the soil may change from the plastic to the semisolid or from the semisolid to the solid with small change in moisture content.

SOIL IDENTIFICATION CHART SHOWS BASIC RELATIONS

The chief value of the soil constants as a means of identifying subgrade soils lies in the relations existing between them rather than in the magnitudes of the individual constants, considered separately. The use of these interrelationships, combined with the values of the constants themselves, as a basis for the construction of the soil identification chart is a distinctive feature of the procedure developed by this bureau in its subgrade studies.

The four graphs of Figure 36 constitute the soil identification chart. They show the relations which have been found experimentally to exist between the liquid limit and four other test constants, the plastic limit, the shrinkage limit, the centrifuge moisture equivalent, and the field moisture equivalent. In the paragraphs which follow the test constants are defined, their significance is explained, and the relations which form the basis of the soil identification chart are developed.

Critical moisture.—The deformations of either confined or unconfined soil samples under constant load increase with increase of moisture content at a consistent rate until a given moisture content known as the critical moisture is reached. When the moisture content is increased above this value the deformations of the samples increase at a very much greater rate than for similar moisture increases below the critical moisture. This fact is illustrated in Figure 37, which is the reproduction of a curve published previously in PUBLIC ROADS (*12*).

It will be noted in this figure that, for a load of 5.6 pounds per square inch, the deformation increases at the rate of about 0.00026 inch for each 1 per cent increase in moisture content below 26.7 per cent, the critical moisture. For increase in moisture content above 26.7 per cent the deformations increase at the rate of 0.022 inch for 1 per cent increase in moisture content.

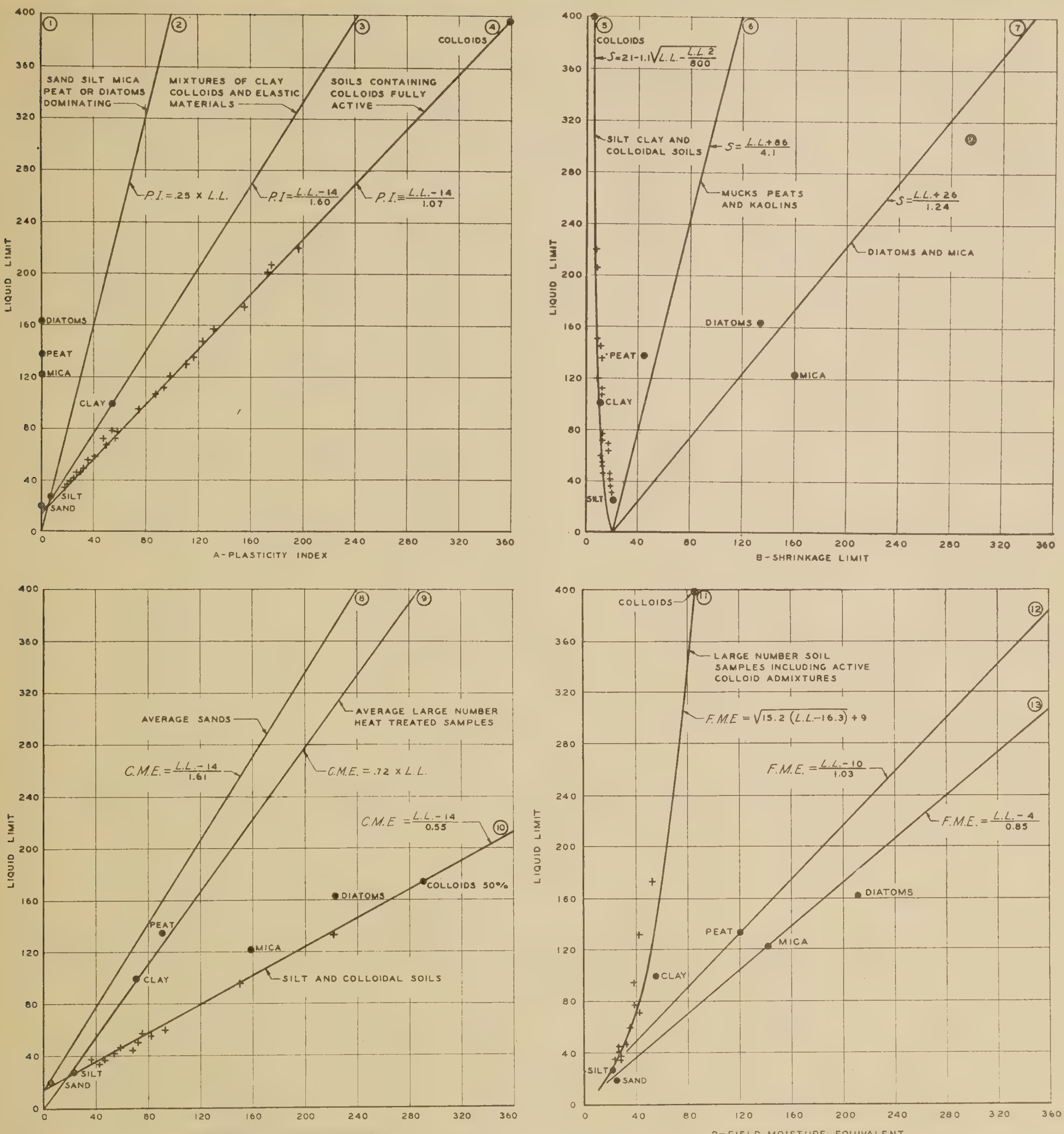


FIGURE 36.—THE SOIL IDENTIFICATION CHART: RELATIONS EXISTING BETWEEN LIQUID LIMIT AND OTHER TEST CONSTANTS

The ratio of applied load to corresponding deformations for different soils is not related to the critical moistures of these soils. As a consequence, therefore, the relation between the critical moisture of one soil and the critical moisture of another soil is not indicative of the relative supporting powers of the two soils. Furthermore, the degree of support indicated by the load-deformation relationship of soils at the critical moisture content varies widely in different soils.

The critical moisture is approximately equal to the plastic limit of cohesive soils and to 75 per cent of the liquid limit of cohesionless soils, which do not have plastic limits (*12*). The determination of the critical moisture is not a routine test. It assists in the explanation of the significance of the plasticity tests, and is of considerable practical importance, because it indicates the extent to which the moisture contents of soils must be reduced in order that they may be stabilized by drainage.

Plastic limit.—This constant is defined as the lowest moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil can be rolled into threads one-eighth inch in diameter without the threads breaking in pieces.

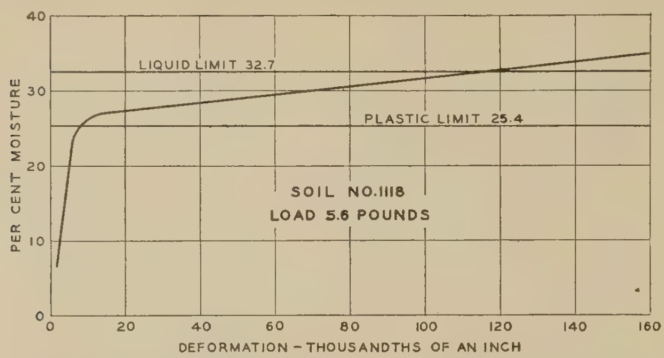


FIGURE 37.—DEFORMATION OF SOIL SPECIMEN WITH INCREASE IN MOISTURE CONTENT. NOTE SUDDEN INCREASE IN RATE OF DEFORMATION AS CRITICAL MOISTURE IS REACHED. PLASTIC LIMIT AND LIQUID LIMIT ARE ALSO MARKED ON THE CURVE

Figure 38 shows the nature of the test for determination of the plastic limit. The upper sample, having a moisture content above the plastic limit, can be rolled into threads less than one-eighth inch in diameter without crumbling under the pressure exerted by the hand. The pressure required to deform the threads varies widely with the character of the soil. The lower part of the drawing shows a soil thread which has crumbled because the moisture content of the soil has been reduced by evaporation to the plastic limit or below.

The prime importance of the plastic limit with respect to this discussion is the fact that it furnishes part of the data required for computing the plasticity index. The following significant relationships should also be noted:

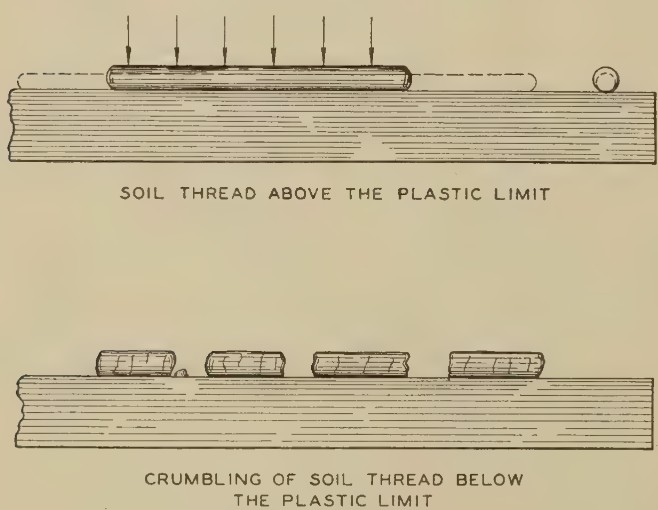


FIGURE 38.—PHENOMENON OCCURRING DURING THE PLASTIC LIMIT TEST

The plastic limit (*a*) equals approximately the moisture content at which a minimum capillary pressure of 2.5 kilograms per square centimeter acts upon the soil sample; (*b*) equals the moisture content at which the coefficient of permeability of homogeneous clays becomes practically equal to zero; (*c*) equals the moisture content above which water evaporates about 4 per cent faster from a clay sample than from the free water surface; (*d*) equals the moisture content at which the speed of evaporation starts to decrease; (*e*) equals the moisture content below which the physical properties of water are no longer identical with those of free water (*12*).

Sand, mica, diatoms, and peat have no plasticity and therefore do not have plastic limits. Silts occasionally may have plastic limits. Clay and colloids have plastic limits. Of the representative subgrade soil constituents referred to in this report (p. 119), the silt has a plastic limit equal to 20, that of the clay is 46, and that of the colloids is 45.

Liquid limit.—The liquid limit is defined as that moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil will just begin to flow when lightly jarred 10 times. According to this definition soils at the liquid limit have a very small but definite shear resistance, which may be overcome by the application of very little force. This resistance, it should be noted, is by definition practically constant for all soils when at the liquid limit, as contrasted with the variable support furnished by different soils when at the plastic limit. At the liquid limit the cohesion is practically equal to zero (*12*).

The nature of the liquid limit test is indicated in Figure 39. The soil sample is placed in a porcelain

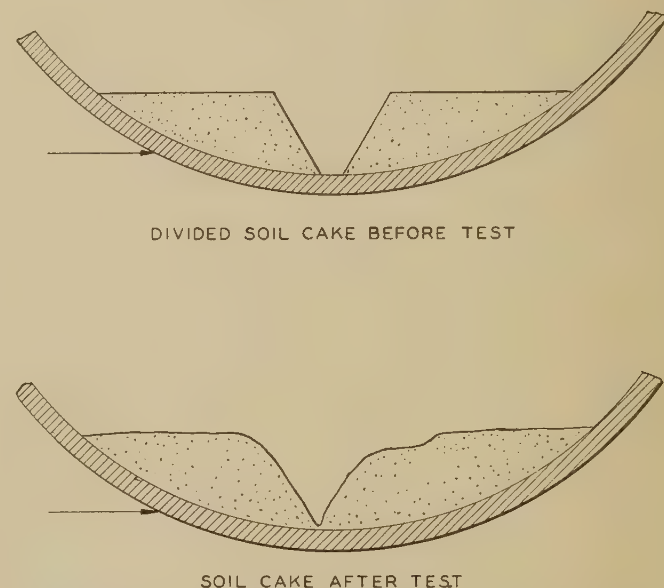


FIGURE 39.—PHENOMENON OCCURRING DURING THE LIQUID LIMIT TEST

evaporating dish about 4½ inches in diameter, shaped into a smooth layer about three-eighths inch thick at the center, and divided into two portions by means of a grooving tool. The dish is held firmly in one hand and tapped lightly 10 times against the palm of the other hand. If the lower edges of the two soil portions do not flow together as shown in the lower part of Figure 39, the moisture content is below the liquid limit. If they flow together before 10 blows have been struck, the moisture content is above the liquid limit. The test is repeated, with more or less moisture present, as the case may be, until a condition is reached where the two edges exactly meet after 10 blows have been struck. The arrows indicate the lateral flow, or shear failure, caused by the 10 blows.

The force created in the soil during the test is a function of the specific gravity of the soil particles combined with full or partial hydrostatic uplift, depending upon whether the soil is of the expansive or nonexpansive type.

Liquid limits of nonexpansive cohesionless soils indicate the moisture content required to lubricate the grain surfaces sufficiently to cause flow under the prescribed force. Liquid limits of expansive cohesionless soils indicate the degree of expansion required to reduce

to practically zero the cohesion furnished by capillary pressure, skin friction, etc. Liquid limits of expansive cohesive soils indicate the degree of expansion required to reduce to practically zero the true cohesion of the soil particles in addition to that furnished by the capillary pressure, skin friction, etc.

In this way the liquid limit serves to distinguish (*a*) sands, with respect to the resistance which they furnish

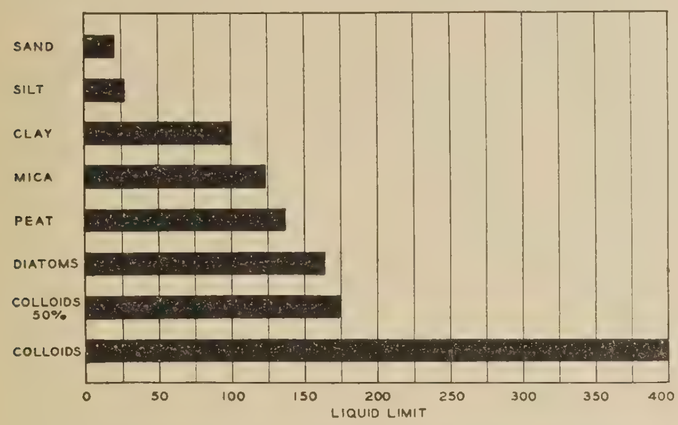


FIGURE 40.—LIQUID LIMITS OF REPRESENTATIVE SOIL CONSTITUENTS

to flowing, and (*b*) all other soils with respect to the relative volume of the pores of capillary dimension which these soils possess. The wide range in the liquid limits of different soils is illustrated by Figure 40, which show graphically the liquid limits of the representative soil constituents.

The liquid limits of the uniform subgrade groups may be stated approximately as follows:

Group A–1.—Generally greater than 14 and less than 25.

Group A–2.—Generally greater than 14 and less than 35.

Group A–3.—Varies in value from slightly less than 10 to slightly more than 35. Small liquid limits, such as 10 to 14, signify the beach and other rounded sand grains which, when sufficiently wetted, will slide over each other, i. e., flow, because of the lubrication of their surfaces. Grains uniform in size and perfectly spherical in form would probably flow without being lubricated. An abnormally large liquid limit, 30 to 35 in the case of sands, signifies high resistance to sliding furnished by a high degree of surface roughness or angularity of grain.

Group A–4. Generally greater than 20 and less than 40.

Groups A–5, A–6, and A–7.—Usually greater than 35.

Group A–8.—Likely to be greater than 45.

Plasticity index.—This term is defined as the difference between the liquid limit and the plastic limit. Plasticity indices equal to zero designate nonplastic soils, i. e., those which have no plastic limits.[8] At the plastic limit the soil particles may be considered as having acquired a degree of lubrication sufficiently high to permit them to slide over each other when loaded, although still possessing cohesion in appreciable amount. At the liquid limit, according to definition, the soil particles are separated to such an extent that practically no cohesion exists between them.

It follows, therefore, that the difference between the plastic limit and the liquid limit indicates the increase in moisture content required to increase the thickness of the water films separating the soil particles to a degree such that the cohesion existing between them is reduced practically to zero. Thus the plasticity index may be considered as a measure of the cohesion possessed by the soil.

[8] Only a very few soils of the thousands tested in the subgrade laboratory of the Bureau of Public Roads have had plastic limits equal to the liquid limits.

As stated above, this cohesion consists of two parts: (*a*) That furnished by capillary pressure, skin friction, etc., and (*b*) that furnished by the true cohesion (molecular attraction) of the soil particles. To illustrate, assume that a pebble is first immersed and then removed from water. The adhesion existing between the surface of the pebble and the water particles in intimate contact with it is very high, but decreases rapidly in amount as the distance separating the water molecule from the pebble surface increases. Consequently the water molecules separated farthest from the pebble surface flow off under the force of gravity.

Two soil particles may be held together by both the adhesion possessed by water for the surface of the soil particles and the cohesion existing between water molecules. With increasing thickness of water film separating the two soil particles the attraction which tends to hold them together rapidly decreases to the vanishing point.

True cohesion existing between soil particles, like the adhesion existing between water molecules and the pebble surface, decreases rapidly in degree as the distance separating the soil particles increases. When the two soil particles possess true cohesion they are held together by a force exceeding that furnished by the molecular attraction of water. Consequently, the thickness of water film required to overcome the true cohesion existing between soil particles is greater than that required to overcome the adhesion due to capillary pressure, skin friction, etc.

If two glass plates are wetted with a small amount of water and pressed together they can not be pulled apart without the application of an appreciable external force. A very slight force, however, may serve to cause one plate to slide upon the other. This condition is analogous to the moisture content of the soil when at the plastic limit. By increasing the thickness of water film separating the two plates, one may be caused by only its own weight to separate from the other. This is analogous to the moisture content of the soil when at the liquid limit. The increase in thickness of water film required to produce the change from the sliding to the separated state represents that portion of the plasticity index required to equalize only the cohesion furnished by the molecular attraction existing between the water and the surfaces of the glass plates.

If, before being wetted and pressed together, the glass plates had been coated with a gluey colloid, the thickness of water film required to produce the change from the sliding to the separated state would of necessity have been larger than if the plates had not been so coated. The difference between the amount of water required to change the coated and the uncoated plates from the sliding to the separated state may be regarded as analogous to the amount of water required to overcome the cohesion possessed by the colloidal particles.

It was stated above that the critical moisture of cohesionless soils equals 75 per cent of the liquid limit. It follows that 25 per cent of the liquid limit may be assumed to indicate the amount of water required to overcome the cohesion furnished by the capillary pressure, skin friction, etc., in soils. If the same principle is applied to cohesive soils, it may be assumed that that portion of the plasticity index exceeding 25 per cent of the lower liquid limit is the amount of water required to overcome the true cohesion existing between the individual soil particles. Consequently, the quantity obtained by subtracting 25 per cent of the liquid limit

from the plasticity index, indicates the relative amounts of true cohesion possessed by the surfaces of the soil particles. Thus the relation existing between the liquid limit and the plasticity index may serve very well to disclose certain characteristics of the soil.

Figure 36, A, illustrates relations existing between the plasticity index and the liquid limit. The large dots denote the relations given in Table 5 for representative soil constituents. Curve 1, indicating a plasticity index of zero, is characteristic of sand, mica, peat, and diatoms. Curve 2 represents the relation, plasticity index = 0.25 × liquid limit. This relation is characteristic of soils containing sand, silt, mica, peat, or diatoms in dominating amounts. Curve 3 indicates the statistical relationship given by tests of a very large number of samples of soils containing clay, colloids, and elastic materials such as peat or mica, and also soils containing colloids not fully active because of either heat treatment or state of flocculation. Curve 4 shows the relation given by compressible mixtures of fully active colloids. The crosses adjacent to curve 4 denote results given by tests of soil samples containing admixtures of bentonite, 1.5 to 50 per cent.

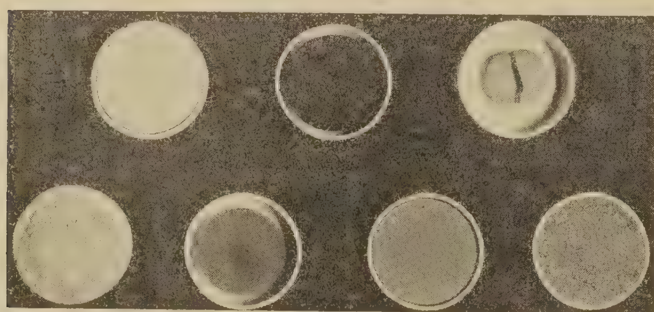


Figure 41.—Shrinkage of Representative Soil Constituents. Top Row, Left to Right, Diatoms, Peat, Colloids; Bottom Row, Left to Right, Mica, Clay, Silt, Sand

The members of the uniform subgrade groups generally have plasticity indices as follows:

Group A-1.—Usually less than 8 and generally related to their liquid limits according to the relation shown by curve 2.

Group A-2.—Usually less than 15.

Group A-3.—Have no plasticity; therefore, do not have plasticity indices.

Group A-4.—Less than those indicated by curve 3.

Group A-5.—Seldom greater than those indicated by curve 3.

Group A-6.—Approximately equal to those indicated by curve 4.

Group A-7.—Generally greater than those indicated by curve 3 and smaller than those indicated by curve 4.

Group A-8.—Generally less than those indicated by curve 3.

Shrinkage limit and shrinkage ratio.—The mechanics of soil shrinkage was described in Part I of this report (pp. 14 and 15) and will not be repeated here. The shrinkage limit is defined as the moisture content, expressed as a percentage of the dry weight, required to fill the pores of a soil sample which has been dried to constant weight from a moisture content sufficient in amount to fill the soil pores completely. Those soils which shrink during this drying process are referred to as possessing significant shrinkage limits. The shrinkage limits of nonexpansive soils such as sand, etc., which do not shrink during this drying process may be computed, as shown later, when the specific gravities of the soils are known. Such shrinkage limits are termed "theoretical" to distinguish them from the "significant" shrinkage limits. Sand and mica have theoretical and the other constituents have significant shrinkage limits. In general, only the significant limits are given in routine test reports.

Figure 41 illustrates the shrinkage of representative soil constituents. The pats shown were dried to constant weight from a moisture content slightly above the liquid limit. The container in which the pat rests represents the original volume of the pat.

The capillary pressure per square centimeter exerted upon the surface of the drying soil sample may be computed as follows:

Assume the voids in the soil mass to be of square cross section and to have each a width equal to a. The perimeter of each tube equals $4a$. Since the force exerted by capillary pressure is equal to 0.0764 grams per centimeter (*1*), the force exerted upon each tube, is given by the expression, $4\,a \times 0.0764 = 0.306a$.

If we should assume that the soil surface is completely covered by such openings, the number of openings per square centimeter equals the reciprocal of the area, a^2, of each opening. Consequently, when we designate *S. F.* as the force causing shrinkage in a soil, we have

$$S.\,F. = \frac{0.306a}{a^2} = \frac{0.306}{a}. \quad \text{-------------(14)}$$

According to this relation, illustrated graphically in Figure 42, the pressure exerted upon a soil possessing voids 0.1 millimeter wide equals 30.6 grams per square centimeter and that exerted upon a soil possessing voids 0.001 millimeter wide equals 3,060 grams per square centimeter. This is the explanation of the fact, illustrated in Figures 41 and 43, that colloids upon drying compact the greatest amount, clay a less, and silt a still less amount.

The shrinkage ratio is defined as the volume change, expressed as a percentage of the volume of the dry soil cake, divided by the moisture loss above the shrinkage limit, expressed as a percentage of the weight of the dry cake.

A determination of skrinkage limit and shrinkage ratio is illustrated in Figure 44. As a soil pat consisting of Yaguajay clay was being dried from the wet state, the changes of both volume and moisture content were determined at different times. The results of these observations are plotted as small circles in the figure. The abscissa of any point on the curve denotes volume change expressed as a percentage of the volume of the dry pat; the ordinate denotes moisture content, expressed as a percentage of the weight of the dry pat.

These points, it will be noted, lie practically on a straight line which intersects the zero line at a moisture content of 11.1 per cent. Consequently no volume change will occurr in the soil cake when the moisture content is reduced from 11.1 to 0 per cent, and 11.1 per cent is the shrinkage limit.

As the moisture content was reduced from 107.8 to 11.1 per cent, the pat underwent a volume change equal to 200 per cent of its volume when dry. The shrinkage ratio is therefore obtained by the computation,

$$\frac{200}{107.8-11.1} = \frac{200}{96.7} = 2.07$$

The shrinkage limit, the shrinkage ratio, and the specific gravity are interrelated as follows:

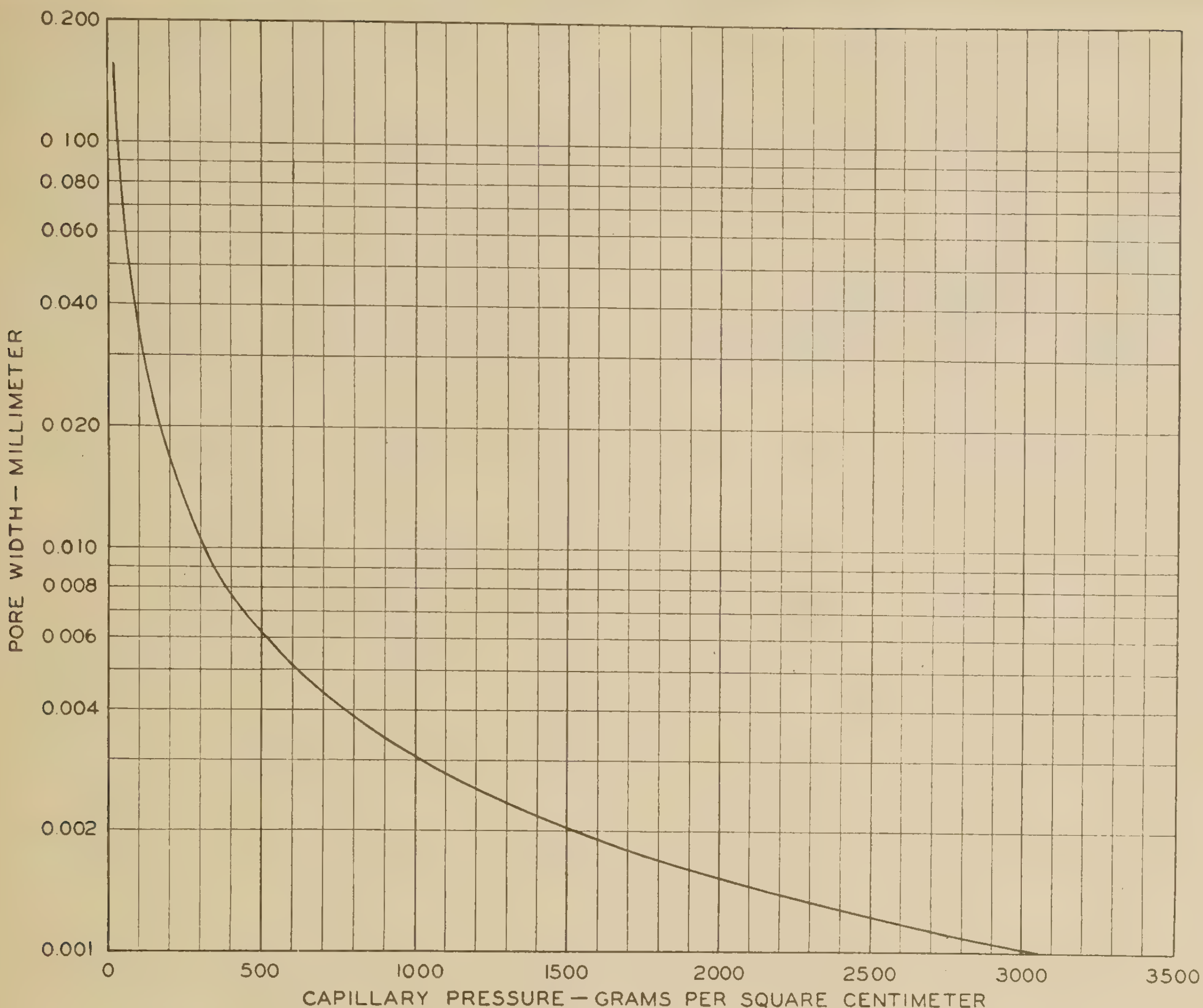


FIGURE 42.—THEORETICAL RELATION BETWEEN PORE WIDTH AND CAPILLARY PRESSURE

Let

S = shrinkage limit.
w = moisture content.
V = volume of wet soil cake.
V_o = volume of dry soil cake.
W = weight of wet soil cake.
W_o = weight of dry soil cake.
R = shrinkage ratio.
G = specific gravity of soil particles.

The total volume change, $V-V_o$, is equal to the moisture loss occurring between the original moisture content, w, and the shrinkage limit, S.

The weight of water lost between these two limits is given by the expression,

$$\frac{w\times W_o}{100}-\frac{SW_o}{100}=\frac{W_o\,(w-S)}{100}$$

If measurements are made in grams and centimeters,

$$\frac{W_o\,(w-S)}{100}=\text{volume of water lost.}$$

$$=V-V_o$$

$$W_ow-W_oS=(V-V_o)\times 100$$

$$W_oS=W_ow-(V-V_o)\times 100$$

$$S=w-\frac{V-V_o}{W_o}\times 100 \quad \text{(15)}$$

And

$$w-S=\frac{V-V_o}{W_o}\times 100$$

The shrinkage ratio, R, defined as the volume change in percentage of the dry volume, divided by the moisture loss above the shrinkage limit in percentage of the dry weight, is given by the equation,

$$R=\frac{\frac{V-V_o}{V_o}\times 100}{w-S}$$

Substituting the value of $w-S$ previously obtained, we have

$$R=\frac{\frac{V-V_o}{V_o}\times 100}{\frac{V-V_o}{W_o}\times 100}=\frac{W_o}{V_o} \quad \text{(16)}$$

The specific gravity equals the weight of the dry soil in grams divided by its true volume in cubic centimeters. The true volume of the dry soil equals the apparent volume, V_o, minus the water content at the shrinkage limit, $\frac{SW_o}{100}$. We have, therefore,

$$G=\frac{W_o}{V_o-\frac{SW_o}{100}}=\frac{1}{\frac{V_o}{W_o}-\frac{S}{100}}=\frac{1}{\frac{1}{R}-\frac{S}{100}} \quad \text{(17)}$$

At one determination of the volume of the Cuban (Yaguajay) soil the weight of the wet pat, W, was 43.74 grams and its volume, V, was 28.04 cubic centimeters. In the dried state the weight of the pat, W_o, was 25.08

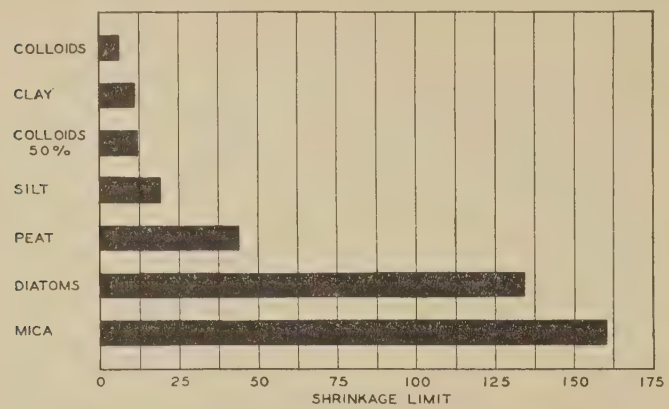


FIGURE 43.—SHRINKAGE LIMITS OF REPRESENTATIVE SOIL CONSTITUENTS

grams and its volume, V_o, was 12.16 cubic centimeters. The value of the moisture content, w, of the wet soil is given by the expression

$$w=\frac{W-W_o}{W_o}\times 100=\frac{43.74-25.08}{25.08}\times 100=74.40 \text{ per cent.}$$

And the shrinkage limit,

$$S=w-\frac{V-V_o}{W_o}\times 100=74.40-\frac{28.04-12.16}{25.08}\times 100=$$

11.1 per cent.

This value checks with that shown in Figure 44.

The volume change which occurred during the loss of 74.4 per cent of moisture, is given by the equation

$$C_o=\frac{V-V_o}{V_o}\times 100=\frac{28.04-12.16}{12.16}\times 100=130.6 \text{ per cent.}$$

This value is shown in Figure 44 by means of a double circle.

The shrinkage ratio $R=\frac{W}{V_o}=\frac{25.08}{12.16}=2.06$ as compared with 2.07, the value obtained from Figure 44.

It will be noted that the shrinkage ratio R is also the apparent specific gravity of the soil. Thus the weight of a cubic foot of the Cuban clay in the dried state equals $62.5\times 2.06=128.75$ pounds.

The specific gravity computation may be subject to appreciable error, since it is based on the assumption that the relation which exists between moisture loss and percentage volume change is as consistent as that indicated in Figure 44. Certain elastic soils are likely to expand during the period when the moisture content is being reduced from the shrinkage limit to zero. Mica is one of these materials and the degree to which it may expand under these conditions is illustrated in Figure 45. This accounts in part for the high shrinkage limit given for mica in Figure 43. Furthermore, the presence of air in the pores of a drying soil may prevent it from shrinking exactly in accordance with the relation shown in Figure 44. Figure 46 shows how different soils may vary in drying from the relation as stated. Table 6 illustrates the variation which may exist between actual and computed specific gravities of soils.

TABLE 6.—*Comparison of specific gravities as obtained by actual determination and by computation from formula 17*

Soil No.	Specific gravity, actual determination	Specific gravity computed from formula	Soil No.	Specific gravity, actual determination	Specific gravity computed from formula
2, 262	2. 72	2. 70	2, 242	2. 640	2. 67
2, 264	2. 70	2. 75	2, 499	2. 815	2. 80
2, 280	2. 59	2. 55	2, 443	2. 752	2. 77
2, 235	2. 50	2. 38	1, 688	2. 716	2. 72
2, 278	2. 64	2. 58	1, 606	2. 740	2. 75
2, 261	2. 62	2. 72	1, 547	2. 653	2. 67
2, 272	2. 65	2. 71	1, 611	2. 612	2. 62
2, 243	2. 71	2. 79	2, 370	2. 715	2. 72
2, 251	2. 54	2. 62	2, 368	2. 660	2. 66
2, 248	2. 63	2. 65	2, 365	2. 640	2. 51
2, 250	2. 71	2. 67			

Figure 36, B illustrates relations existing between the shrinkage limit and the liquid limit. The large dots denote the relations given in Table 5 for the representative soil constituents. Curve 5 is characteristic

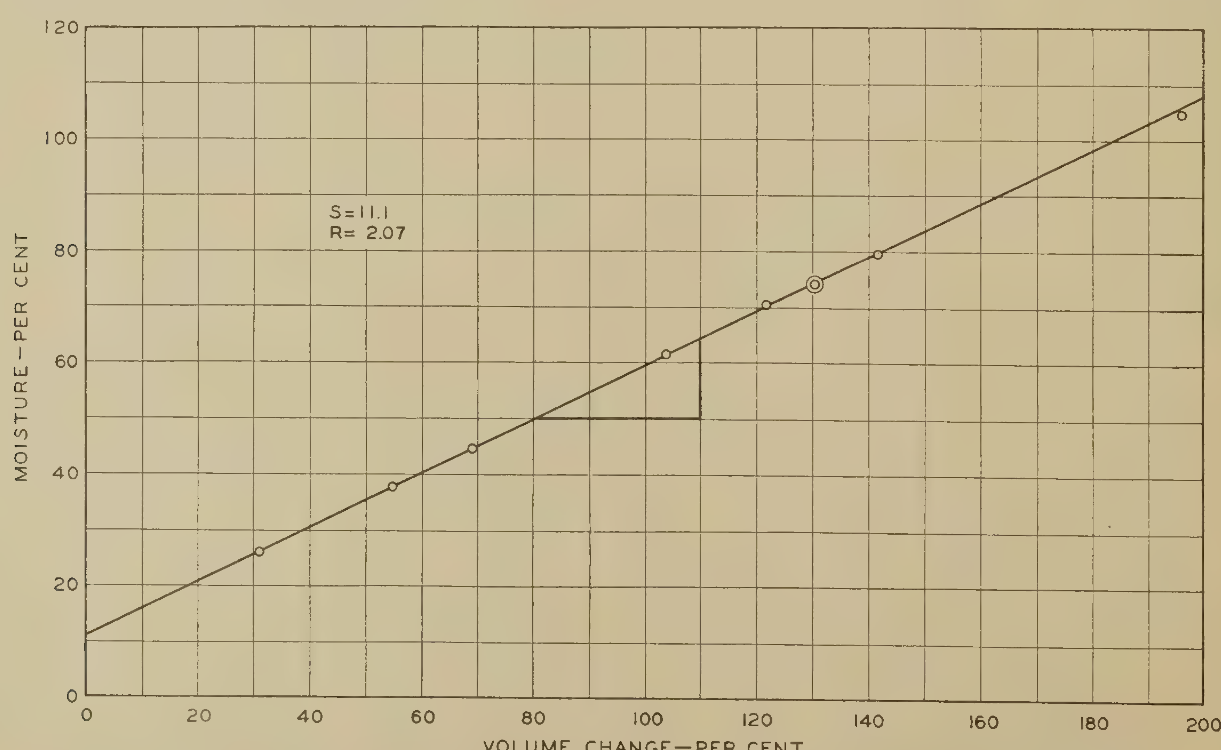


FIGURE 44.—RELATION BETWEEN MOISTURE CONTENT AND VOLUME CHANGE FOR YAGUAJAY CLAY

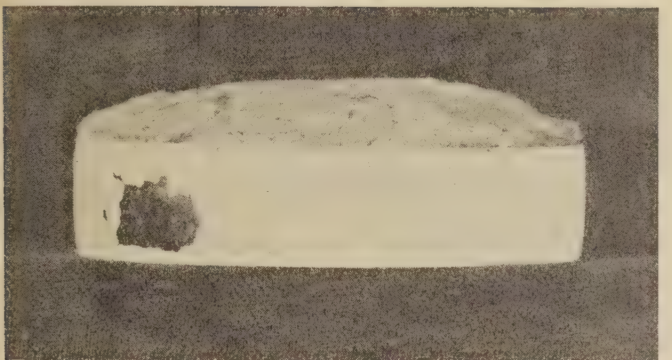


FIGURE 45.—SIDE VIEW OF MICA PAT SHOWING EXPANSION ON DRYING TO CONSTANT WEIGHT FROM A MOISTURE CONTENT SLIGHTLY ABOVE THE LIQUID LIMIT. THE MICA JUST FILLED THE CUP WHEN WET

of silt, clay, and colloidal soils. It also indicates approximately the average relation given by tests of a large number of natural soil samples (*21*). The small crosses adjacent to this curve represent results of tests performed on natural soil containing bentonite in admixtures varying between 1.5 and 50 per cent. Curve 6 indicates the moderately high shrinkage limits characteristic of mucks, peats, and kaolins. Curve 7 represents diatoms and mica, which have high shrinkage limits.

Shrinkage limits characteristic of the uniform subgrade groups are as follows:

Group A–1.—Generally greater than 14 and less than 20.

Group A–2.—May be either theoretical or significant depending upon other constants. Not likely to exceed 25 when significant.

Group A–3.—No significant shrinkage limit.

Group A–4.—Generally less than 25. Increase in expansive properties of members of this group indicated when shrinkage limits exceed 20 and approaches relationship represented by curve 6.

Group A–5.—Generally greater than 30 and greater than 50 for very undesirable members of this group. May approach relation indicated by curve 6 for silts containing peat and approach relation indicated by curve 7 for soils containing either diatoms or mica in appreciable amount.

Group A–6.—Not likely to exceed in appreciable amount values represented by curve 5.

Group A–7.—For flocculated varieties of this group, may slightly exceed values given by curve 5. For varieties containing mica, peat or diatoms, may exceed very appreciably values indicated by curve 5, but generally less than those indicated by curve 6. Soils of this group subject to frost heave have the higher shrinkage limits.

Group A–8.—Generally in neighborhood of values indicated by curve 6, and seldom greatly in excess of those values.

Centrifuge moisture equivalent.—This constant is defined as the moisture content, expressed as a percentage of the weight of the oven-dried soil, retained by a soil sample after first being soaked in water for 6 hours, then drained in a humidifier for 12 hours, and finally centrifuged under an acceleration of 1000 × gravity for 1 hour.

The action which takes place is illustrated in Figure 47. Water is forced outward through the bottom of the cup under the influence of two forces, the centrifugal force acting on the water and the pressure which the soil particles exert on one another. The centrifugal force acting on the water is proportional to the distance from the surface of the sample. Since the acceleration is $1000g$ the pressure at a distance y will be equal to y kilograms per square centimeter. The pressure which the individual particles exert upon each other in the direction of the axis of the cup is a function of the specific gravity of the particles, the distance from the surface, and the extent of hydrostatic uplift. If the

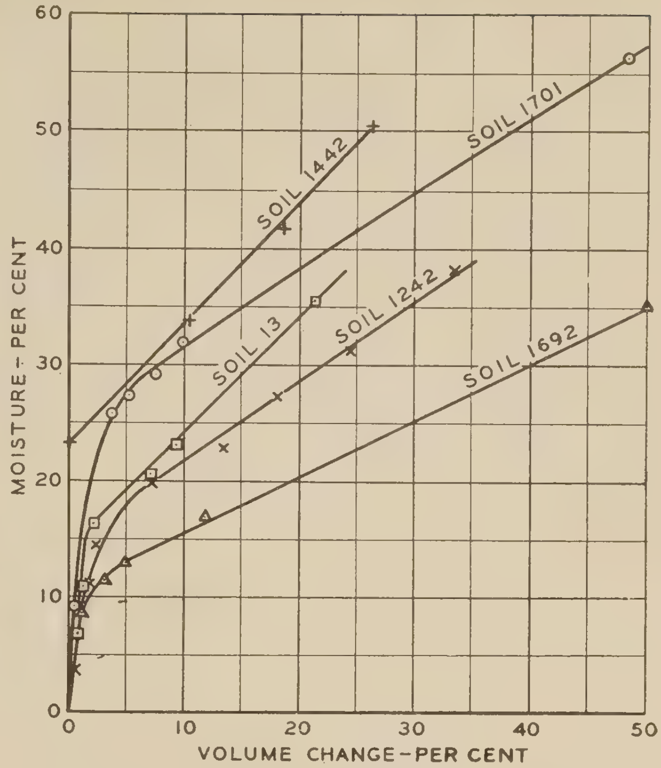


FIGURE 46.—TYPICAL CURVES SHOWING VARIATION OF VOLUME CHANGE WITH MOISTURE CONTENT

soil particles are not surrounded by water, the hydrostatic uplift is negligible, and we may define the pressure on the particles as Py kilograms per square centimeter, where P is a function of the specific gravity. This is the case with nonexpansive soils. In the case of expansive soils the hydrostatic uplift is much greater. If we assume it as full, the pressure becomes $(P-1)y$, or, at the bottom of the cup, $(P-1)h$.

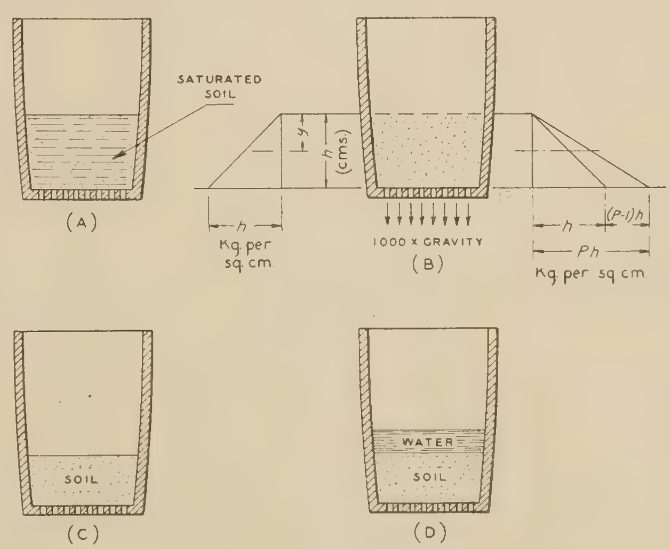


FIGURE 47.—PHENOMENON OCCURRING DURING THE CENTRIFUGE MOISTURE EQUIVALENT TEST

This combination of forces results in pressures which are equivalent to an average pressure on the sample of about 2 kilograms per square centimeter. As the intimacy of contact between soil particles is increased, the tendency is for water to be squeezed from the sample at both top and bottom. The tendency for water to be forced to the top is resisted by the centrifugal force acting on the water. This force is in turn opposed by

the frictional resistance to flow offered by the surfaces of the soil pores and the capillarity of the soil, both of which increase as the soil mass decreases in volume during the test. If the resisting force is greater than the centrifugal force water remains at the top of the sample, producing the condition called waterlogging. The amount of water which is forced through the sample and escapes through the outlets at the bottom of the cup is thus dependent on the permeability of the soil.

Thus the centrifuge moisture equivalent (*a*) serves to distinguish soils which are permeable (sand, silt, mica, diatoms, peat or flocculated clay dominating) from those which are impermeable (clay and colloids dominating) when compressed by a centrifugal force equal to about 2 kilograms per square centimeter (*12*); (*b*) serves to disclose to some extent the degree of capillarity possessed by permeable soils; (*c*) furnishes a means of distinguishing permeable soils of the non-expansive from those of the expansive varieties.

The moisture equivalents of permeable soils, for instance, decrease consistently when the sand content of the soils is increased. The shrinkage limits of silt and clay soils having capillarity in appreciable amount increase at a very slow rate with increase in the sand content of the soils until the amount of sand added becomes sufficient to reduce the capillary properties of the soil very appreciably. At this sand content the shrinkage limits of the soils become theoretical instead of significant and increase at an abnormally high rate with further additions of sand. Therefore, by plotting the centrifuge moisture equivalents against the corresponding shrinkage limits of soils to which sand has been added in increasing amounts, the centrifuge moisture equivalent value at which the shrinkage limits suddenly begin to increase is easily determined. This value of the centrifuge moisture equivalent should indicate the degree of capillarity below which expansion and shrinkage become negligible in amount.

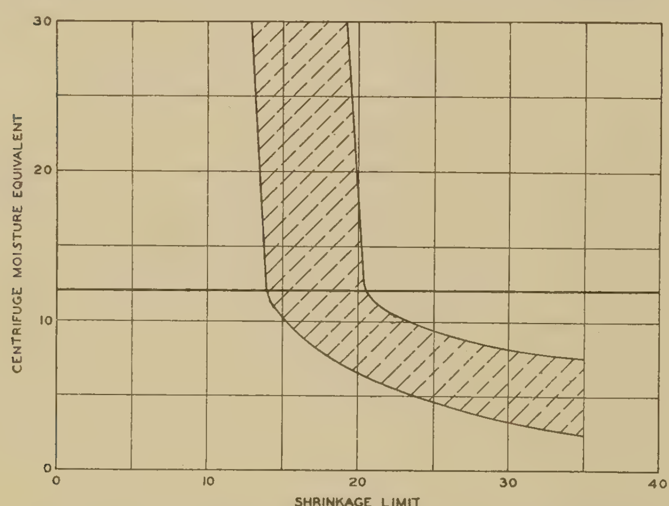


FIGURE 48.—RELATION BETWEEN CENTRIFUGE MOISTURE EQUIVALENT AND SHRINKAGE LIMIT FOR SAND ADMIXTURES

Figure 48 shows the results furnished by a determination of this character. The soil constants were obtained from tests performed upon a number of soils containing sand in amounts varying between 20 and 80 per cent. The results, it will be noted, are grouped in a well defined band which indicates a very pronounced increase in the theoretical shrinkage limits when the amount of contained sand is sufficient to reduce the centrifuge moisture equivalents to less than 12.

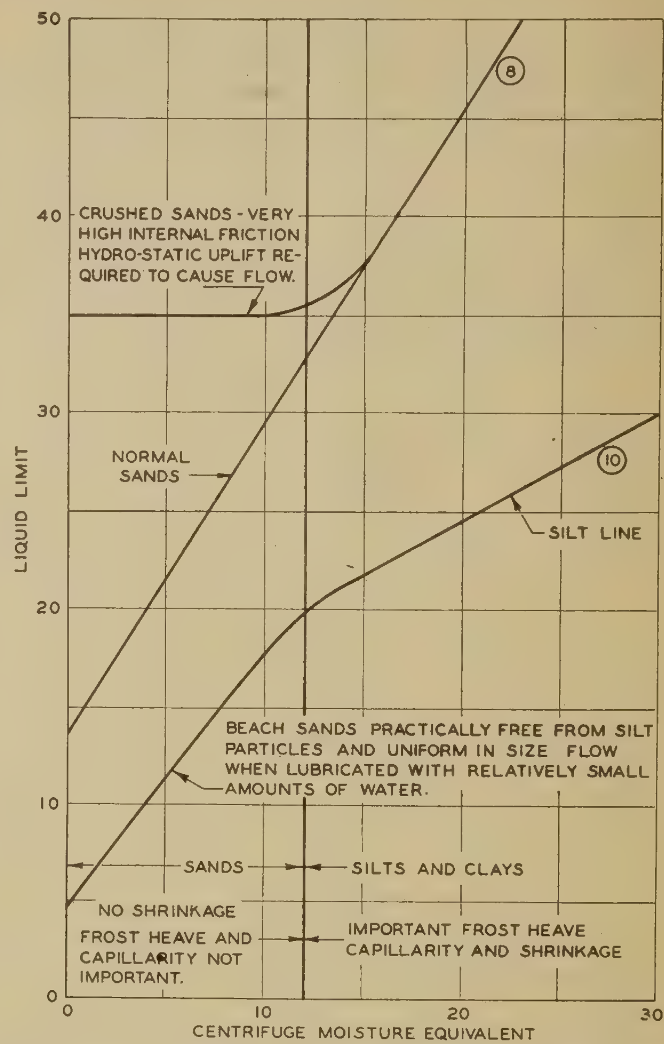


FIGURE 49.—RELATION BETWEEN LIQUID LIMIT AND CENTRIFUGE MOISTURE EQUIVALENT FOR DIFFERENT TYPES OF SAND

Since detrimental frost heave occurs only in soils having capillarity in appreciable amount, it should not occur in soils which have centrifuge moisture equivalents less than about 12. And this seems to be verified by the subgrade surveys of the bureau now in progress. To provide a proper factor of safety, however, materials used for porous (*7*) base courses in practice, should have centrifuge moisture equivalents not exceeding 6 or 8.

The centrifuge moisture equivalents of the representative soil constituents (Table 5) are shown graphically in Figure 36, C. This figure contains also curves 8, 9, and 10, which indicate important relations existing between the centrifuge moisture equivalent and the liquid limit. Curve 8 represents the relation between the liquid limits and the centrifuge moisture equivalents of average sands (absence of hydrostatic uplift). Curve 9 represents the statistical relation between the average liquid limits and centrifuge moisture equivalents obtained from a great number of heat-treated soil samples (*21*). Curve 10 represents the relation between the liquid limits and the centrifuge moisture equivalents of average silt and colloidal soils (particles acting under full hydrostatic uplift).

Members of the uniform subgrade groups may have centrifuge moisture equivalents as follows;

Group A–1.—Seldom appreciably greater than 15.
Group A–2.—Not likely to exceed 25.

Group A–3.—Not likely to exceed 12. In combination with the liquid limit, discloses the relative resistance to flowing possessed by sands equal in degree of capillarity. (See Fig. 49).

Group A–4.—Generally greater than 12, approaching values indicated by curve 10, but not likely to waterlog, although exceptions occur. When greater than the liquid limits in the absence of waterlogging, indicates especially unstable silts.

Group A–5.—Greater than 12 and not likely to waterlog, although exceptions occur. Often has values between curves 9 and 10. May approach those indicated by curve 8 for sand-mica mixtures.

Group A–6.—May approach values indicated by curve 10 for the highly colloidal soils, with values lying between curves 9 and 10 for clay soils containing sand in appreciable amount. Likely to waterlog when exceeding 40.

Group A–7.—Generally greater than values indicated by curve 9 and less than values indicated by curve 10. May not waterlog with centrifuge moisture equivalents as high as 90.

Group A–8.—Generally between curves 9 and 10.

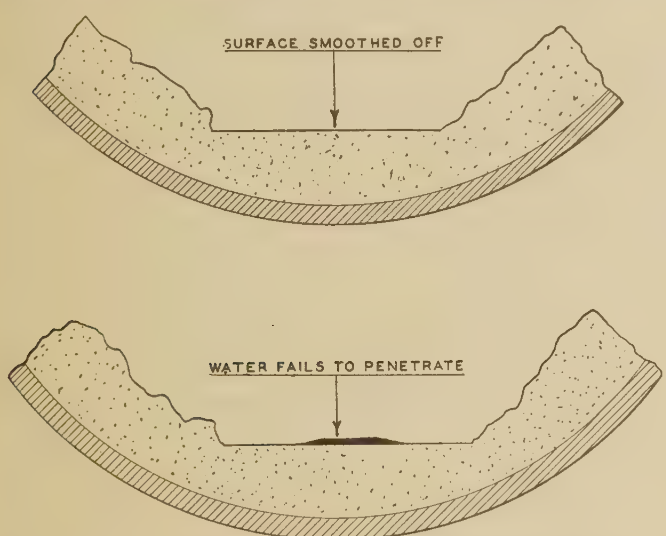


Figure 50.—Phenomenon Occurring During the Field Moisture Equivalent Test

It is interesting to note also that adding sand to the silt and clay mixes, which are represented by curve 10, will change these soils to graded mixes of the Group A–1 or Group A–2 variety and the test relationship will approach that indicated by curve 9. Adding both sand and mica to a silt may cause its centrifuge moisture equivalent to assume the relation to the liquid limit represented by curve 9.

Field moisture equivalent.—This term is defined as the minimum moisture content, expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed, but will instead spread out over the surface and give it a shiny appearance. (See fig. 50.)

The drop of water fails to penetrate the wet and smoothed soil sample (*a*) when the pores of nonexpansive soils (sands) are completely filled, (*b*) when the capillarity of cohesionless expansive soils (diatoms and mica) is completely satisfied, and (*c*) when cohesive soils possess moisture in amount sufficient to cause the smoothed surface of the sample to become impervious. This impervious skin may occur at moisture contents far below those required to satisfy the capillarity of cohesive soils.

That the moisture content at which the impervious skin is formed measures a definite soil property and is not dependent upon the time during which the soil remains wetted is evidenced by the fact that highly colloidal clays, whether wetted for several minutes or for 24 hours, generally resist the penetration of the drop of water at practically equal moisture contents.

Figure 36, D contains curves showing relations existing between the liquid limit and the field moisture equivalent. Curve 11 represents the statistical relationship which was found to exist between the averages of results furnished by tests performed upon a large number of natural soil samples (*21*). This curve, it will be noted, represents also the relation given by results of tests performed upon the soils containing admixtures of active colloids.

The positions of curves 12 and 13 were chosen arbitrarily to represent high and very high field moisture equivalents. The field moisture equivalents of the representative soil constituents (Table 5) are also shown graphically in Figure 36, D. The field moisture equivalent individually and in its relation to the other constants serves to furnish the following supplementary information with respect to the identification of subgrade soils.

Group A–1.—Field moisture equivalent not significant.

Group A–2.—Field moisture equivalent not significant.

Group A–3.—Field moisture equivalent indicates the porosity of these cohesionless materials when completely saturated; in combination with the liquid limit discloses the degree of saturation required to cause sands to have a very small shear resistance.

Group A–4.—When approximately equal to or larger than centrifuge moisture equivalents the field moisture equivalents indicate presence of expansive properties in detrimental amounts.

Group A–5.—Field moisture equivalents may approach values indicated by curve 12 for silts containing peat in appreciable amount and those indicated by curve 13 for highly elastic silts containing mica or diatoms in appreciable amount. May not exceed those indicated by curve 11 for kaolins possessing good binder properties.

Group A–6.—May approach values indicated by curve 11 generally, but smaller when the grading of the colloidal clay soils of this group is such as to cause smoothed surface of soil when wetted to become highly impermeable.

Group A–7.—Soils of this group either flocculated or containing organic matter partially decomposed into the colloidal state

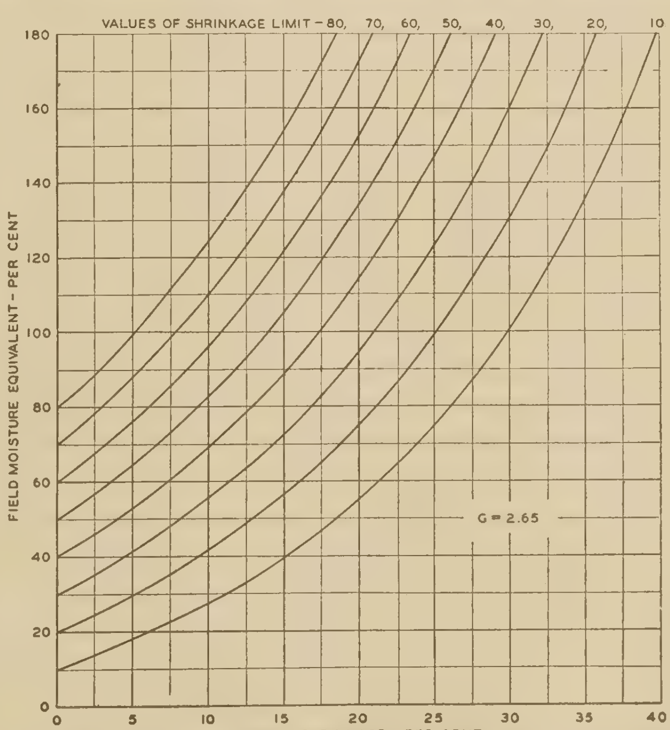


Figure 51.—Chart for Estimating Shrinkage Limit from Values of Field Moisture Equivalent and Lineal Shrinkage

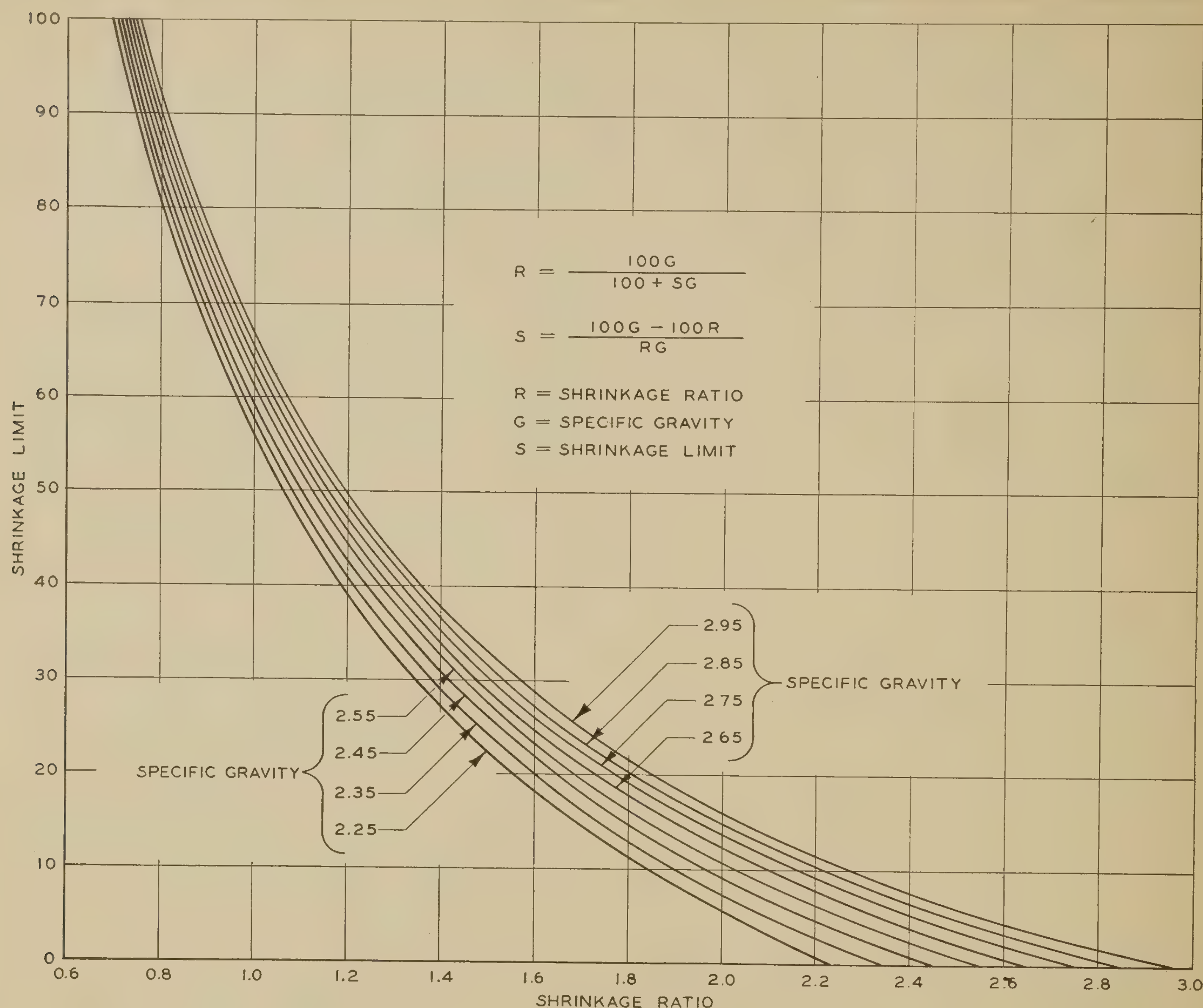


FIGURE 52.—RELATION BETWEEN SHRINKAGE LIMIT AND SHRINKAGE RATIO FOR DIFFERENT VALUES OF SPECIFIC GRAVITY

may have, as in the case of the Group A–6 soils, shrinkage limits approaching those represented by curve 5. Their field moisture equivalents, however, are likely to be appreciably greater than those indicated by curve 11.

Group A–8.—Same as group A–5.

Volumetric change.—Only in certain instances does the degree to which soils may shrink when dried out from an arbitrary wet state furnish information with respect to the identification of subgrade soils supplementary to that furnished by the constants discussed above. Volumetric changes from the field moisture equivalent are now computed only to determine whether those of graded materials and silty clays are larger or smaller than about 17. This limit, according to computation, is equivalent to a lineal shrinkage of 5, a value which has been established by A. C. Rose and C. H. McKesson (PUBLIC ROADS, August, 1924, September, 1925, and September, 1927), as representing the maximum degree of shrinkage properties which may be possessed by good soil mortars or stable subgrade soils. Thus, in certain instances the volumetric change assists in distinguishing the Group A–6 and Group A–7 soils which are inclined to shrink in appreciable amount (volumetric change approximately equal to or greater than 17) from the Group A–2, A–4, or A–5 varieties in which shrinkage is not important.

Let

F. M. E. = field moisture equivalent,
G = specific gravity of soil particles,
S = shrinkage limit,
R = shrinkage ratio.

The volumetric change, C_f, is given by the formula

$$C_f = (F.\,M.\,E. - S) \times R \qquad (18)$$

We have

$$G = \frac{1}{\frac{1}{R} - \frac{S}{100}} \qquad (17)$$

Hence,

$$R = \frac{100G}{100 + GS} = \frac{1}{\frac{1}{G} + \frac{S}{100}};$$

And, by substitution,

$$C_f = \frac{F.\,M.\,E. - S}{\frac{1}{G} + \frac{S}{100}} \qquad (19)$$

No computation of the volumetric change is required for the average clay soils of the Group A–6 or Group A–7 variety when the shrinkage limits do not exceed those shown by curve 5 and the field moisture equivalents are not less than those indicated by curve 11. With a liquid limit of 35, for instance, such soils have shrinkage limits not exceeding 15 and field moisture equivalents not less than 26. By substitution of these values in equation 18, we find that the volumetric change, with an average R equal to 1.8, exceeds 20, and for this value the lineal shrinkage is larger than 5.

Lineal shrinkage.—The volumetric change, C_f, is expressed as a percentage of the dry volume of the soil cake. The lineal shrinkage, *L. S.*, expressed as a percentage of the length of wet soil bar, is defined by the formula

$$L.\ S. = 100\left[1 - \sqrt[3]{\frac{100}{C_f + 100}}\right] \text{-------------} (20)$$

The lineal shrinkage, as such, possesses no more significance than the volumetric change. However, the lineal shrinkage combined with the field moisture equivalent offers a means of estimating the shrinkage limit which may be used when the first two values are known and the shrinkage limit has not been determined by test.

The relation between lineal shrinkage, field moisture equivalent, and shrinkage limit computed by equations 19 and 20 for soils having a specific gravity of 2.65 is shown graphically in Figure 51. According to this figure a soil having a lineal shrinkage of 17 combined with a field moisture equivalent of 82 would have a shrinkage limit of approximately 30.

The relation between shrinkage limit and shrinkage ratio for specific gravities varying from 2.25 to 2.95 is shown in Figure 52.

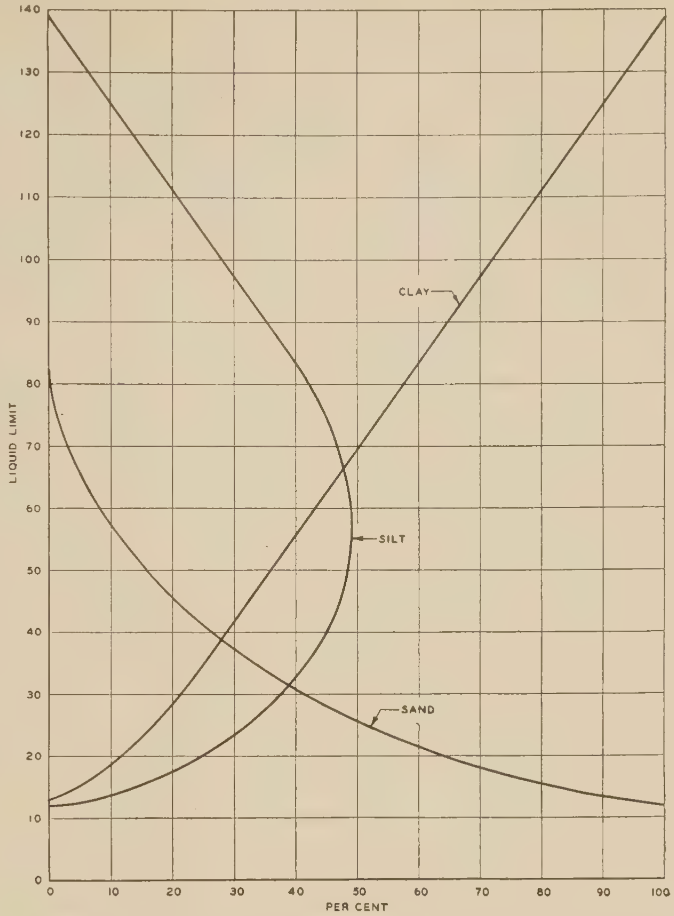


FIGURE 53.—AVERAGE MECHANICAL ANALYSIS OF SOILS WHOSE TESTS ARE RELATED ACCORDING TO CURVES 3, 5, 9, AND 11, FIGURE 36

TEST CONSTANTS AND MECHANICAL ANALYSES STATISTICALLY RELATED

In addition to knowing the interrelationships between the test constants it may prove helpful also to have some conception of the average relations existing between the clay, silt, and sand contents of soils and their test constants.

The relations represented by curves 3, 5, 9, and 11 (fig. 36) are, in general, those which have been reported previously as "statistical" relationships between the averages of large numbers of individual soil tests (*21*). The average mechanical analysis of soils whose tests are related according to these statistical laws is shown in Figure 53 as a function of the liquid limit.

Both the grading represented in Figure 53 and the constants represented by curves 3, 5, 9, and 11 may be considered as characteristic of "average soils," and this fact may serve as a basis for estimating the relative degree to which particular soils possess certain characteristics.

An "average" or "statistical" soil containing 72 per cent clay and no sand has, according to Figures 53 and 36, constants as follows: Liquid limit, 100; plasticity index, 54; shrinkage limit, 11; centrifuge moisture equivalent, 72; and field moisture equivalent, 45. If a soil being investigated contains 72 per cent of clay and no sand, but the constants have the values, liquid limit, 50, plasticity index, 30, shrinkage limit, 15, centrifuge moisture equivalent, 60, and field moisture equivalent, 55, its constants may be expressed as ratios of the constants possessed by the statistical soils.

As the investigations in progress disclose more and more the soil constituents which cause the constants of various soils to differ from the constants of statistical soils, the relations discussed above will serve to reveal more accurately the constituents of which the soils being investigated are composed.

It is interesting to note that the representative soil constituent, clay, in Table 5, is for all practical purposes an "average" soil.

The constants of statistical soils containing both clay and sand in amounts equal to those indicated in Figure 53 are obtained in the same manner as the constants of statistical soils containing clay and no sand.

Thus the constants of a statistical soil containing 36 per cent clay and 16 per cent sand have the following values: Liquid limit, 50; plasticity index, 23; shrinkage limit, 13; centrifuge moisture equivalent, 36; and field moisture equivalent, 32.

It may prove helpful also to be able to estimate what might be termed the average influence exerted by sand admixtures upon the constants possessed by soils. The conversion curves of Figures 54 and 55 furnish a means of making estimates of this character. These curves are based on tests performed upon soils containing sand admixtures in different amounts. The sand content, referred to in these figures as "per cent sand," is expressed as a percentage of the combined weights of both sand admixture and soil.

Estimates furnished by means of conversion curves decrease in accuracy as the amount of the assumed sand admixture is increased. This is due to the fact that the curves indicate the average influence exerted by a number of sands and not the influence exerted by a particular sand. As admixtures of a particular sand

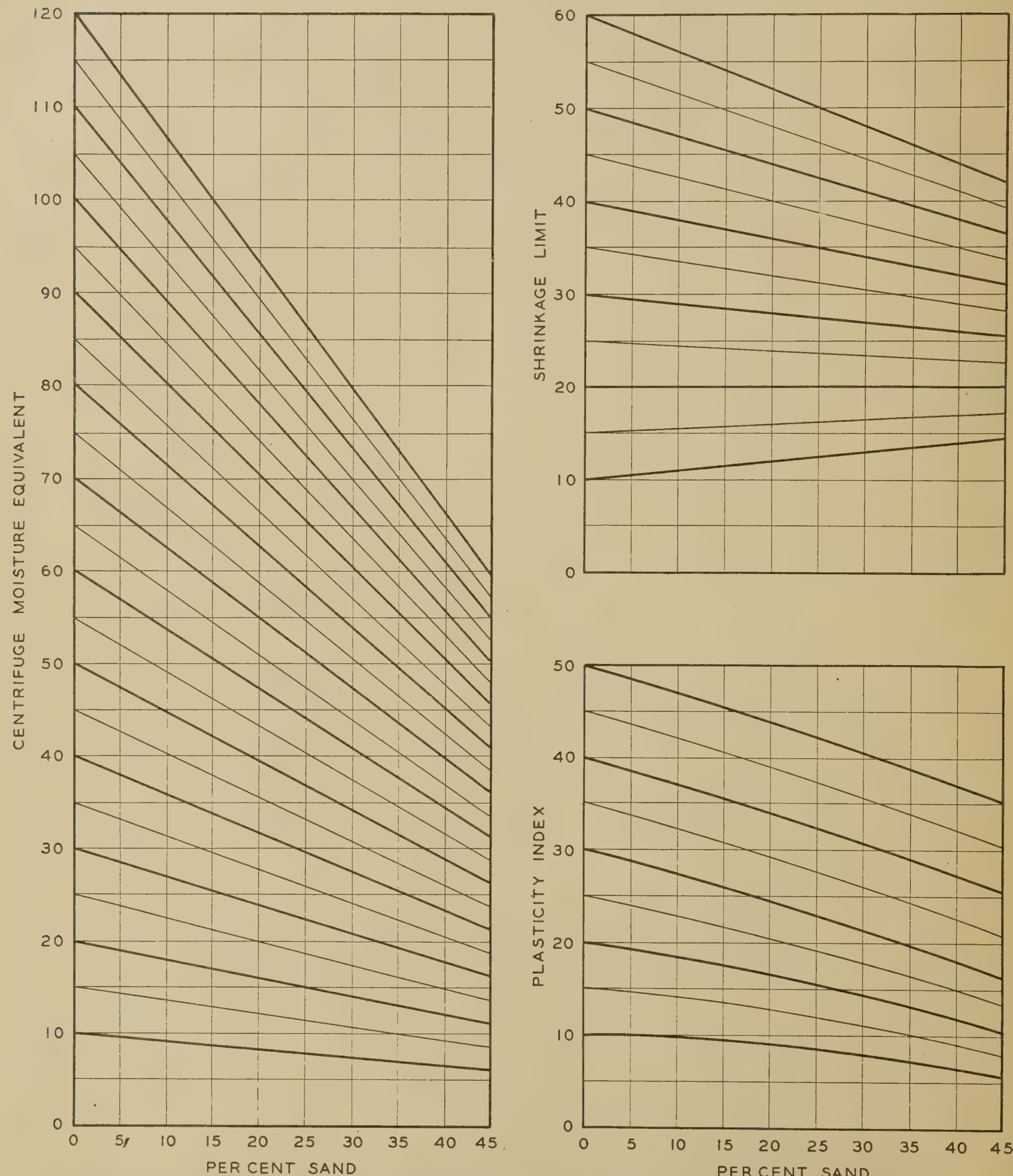


FIGURE 54.—CONVERSION CHARTS FOR DETERMINING EFFECT OF SAND ADMIXTURES ON SOIL TEST CONSTANTS

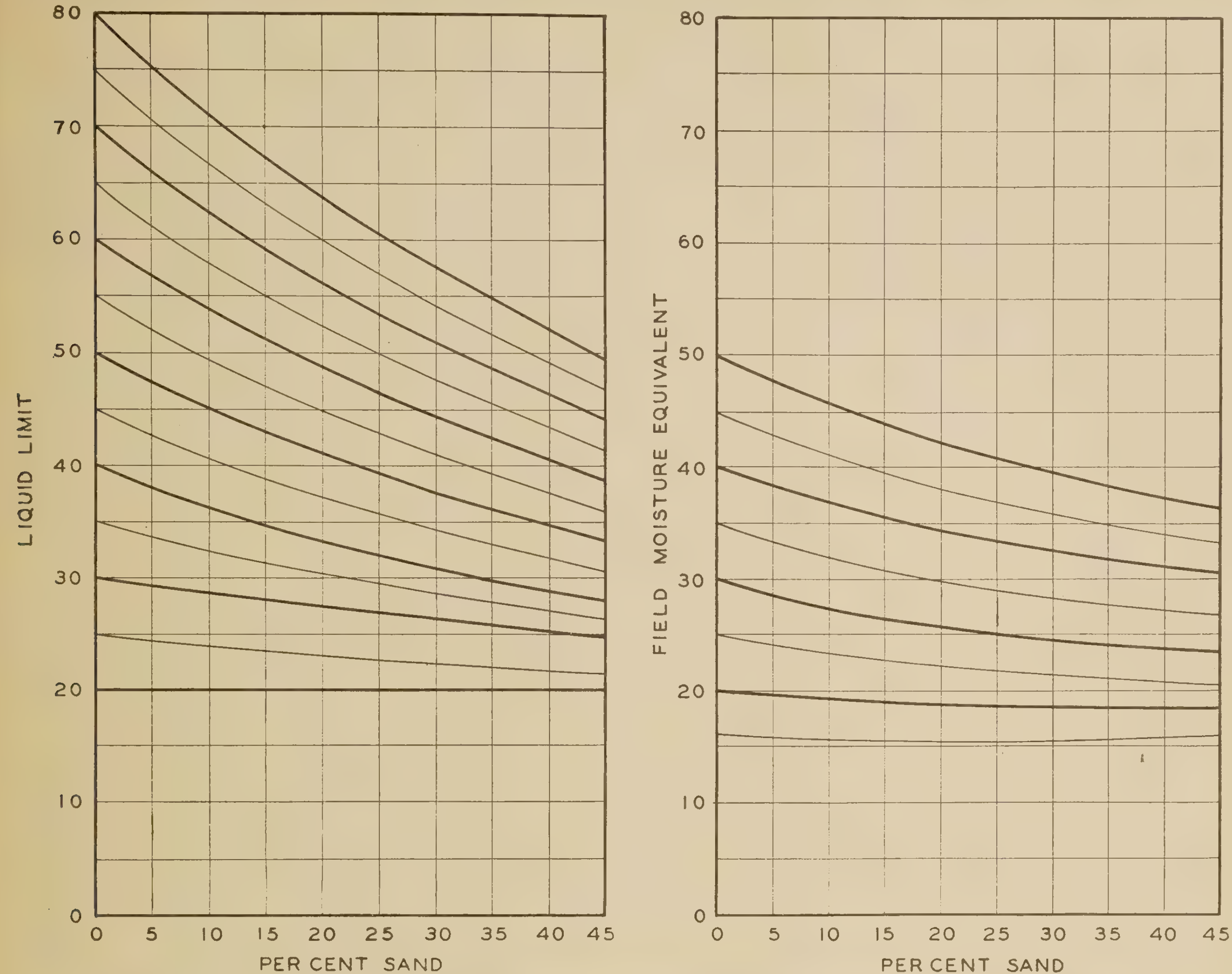


FIGURE 55.—CONVERSION CHARTS FOR DETERMINING EFFECT OF SAND ADMIXTURES ON SOIL TEST CONSTANTS

increase in amount, its individual characteristics, which depend on its grading, the size and shape of its grains, etc., will exert an increasingly important influence upon the test constants of the mixture, and cause them to vary from the estimates furnished by the conversion curves. This is especially true when the sand admixture increases in amount above about 45 per cent. As the sand admixture decreases in amount below about 45 per cent, the physical characteristics of the sand exert less and less influence on the soil test results.

In order to illustrate the use of Figures 54 and 55 let us assume that a soil sample has constants equal to those designated for sample A, Table 7. It is desired to estimate the influence exerted by adding sand in amount equal to 25 per cent of the weight of the soil. The assumed admixture in this case equals $\frac{25 \times 100}{125}$ per cent or 20 per cent of the resulting combination of sand and soil. Consequently, we reduce each constant of sample A by an amount indicated in the corresponding conversion diagram for a sand admixture of 20 per cent. The resulting constants are given opposite sample X in Table 7.

TABLE 7.—*Test constants estimated on the basis of the conversion curves for sand admixtures*

Sample	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
A	45	19	20	1.7	33	30
X	37	16	20	[1] 1.7	26	26

[1] Specific gravity assumed to equal that of sample A. Therefore, with equal shrinkage limits the shrinkage ratios will be equal.

PART III: UTILIZATION OF THE SUBGRADE SOIL IDENTIFICATION CHART

The soil identification chart, shown in revised form in Figure 56 serves a triple purpose. First, it offers a means of identifying those subgrade soils whose performance in service has been learned; second, it assists in predicting the performance of soils comprising the subgrades of roads to be constructed; and, third, it assists in disclosing the influence exerted by either physical or chemical admixtures upon the performance of subgrade soils.

In order to demonstrate how these purposes are accomplished the improvement of soils by means of admixtures is illustrated and a limited number of soils belonging to the different groups are analyzed with respect to their constants. To facilitate this demonstration the basic requirements of good sand clay roads are discussed and both the grading and the constants indicative of soils of the different groups are reviewed.

GRADING OF GOOD TOP SOILS DEPENDS UPON THE CHARACTER OF THE BINDER

Theoretically stable mixtures consist of a well graded coarse material (grains larger than about 0.05 millimeter in diameter) possessing high internal friction, and a binder. The binder, which may be visualized as occupying the sand pores, should have sufficient cohesion to cement the sand grains together; and, upon wetting, the binder should expand in amount just sufficient to close the surface pores and thus prevent water from penetrating and softening the interior of the road surface. When the binder expands an amount greater than that required to close the sand pores, the sand grains are likely to become unseated, thus reducing the stability of the mixture. When the binder does not expand sufficiently to close the sand pores, water may enter and soften the road surface. It follows that the amount of binder required to furnish stable mixtures depends upon the expansion properties of the binder. Binders which are only slightly expansive may be used in an amount sufficient to fill the pores of the sand almost completely. As the expansive properties of the binder become more and more important, the amount used without danger of unseating the sand grains must of necessity become smaller and smaller.

Of two soils whose tendency to shrink or expand are equal, the one having the greater amount of cohesion should be the better binder. Of two soils having equal cohesion, that having the less tendency to shrink or expand should be the better binder, since a greater amount of it can be used than of the more expansive soil.

This theoretical conception that the amount of binder required depends upon the characteristics of the binder is substantiated by Doctor Strahan's (*22*) studies of roads in service. He emphasizes the fact that while soil mixtures having particular gradings are likely to produce stable wearing courses, the gradings are a qualitative rather than a quantitative measure of efficiency. The cohesive and shrinkage properties of the fine material are of utmost importance. Doctor Strahan reports kaolin as being an exceptionally good binder; and according to its constants kaolin possesses properties theoretically required by good binders. These constants are:

Liquid limit	60
Plasticity index	26
Shrinkage limit	36
Shrinkage ratio	1.3
Centrifuge moisture equivalent	49
Field moisture equivalent	36

The liquid limit of 60, for instance, combined with a plasticity index of 26 indicates cohesive properties approaching those of an inert clay (curve 3). The shrinkage limit of 36 equals that of elastic silts and muck (curve 6). The centrifuge moisture equivalent of 49 indicates a water capacity slightly greater than that of average soils (curve 9). The field moisture equivalent of 36 indicates a resistance to water penetration characteristic of colloidal soils (curve 11): and the shrinkage limit being equal to the field moisture equivalent, the lineal shrinkage, like that of sand, is equal to 0.

It is clear that kaolin has both cohesion and water-retentive properties in moderate amount, relatively high resistance to water penetration when at the shrinkage limit, and negligible shrinkage properties. The constants possessed by kaolin may, therefore, serve as a basis for identifying good binders. These characteristics, it will be noted, indicate a plastic variety of the Group A–5 subgrade with an exceptionally low field moisture equivalent.

SUBGRADE SOILS MAY BE IMPROVED BY ADMIXTURES

It is natural that efforts should be made to increase the stability of certain varieties of subgrade soil by admixtures of suitable materials. The success of such efforts depends upon the manner and the extent to which unstable mixed materials differ in character from those which are stable. The changes which admixtures produce in the test constants indicate their effect on the physical properties of the soil.

If, according to mechanical analysis, the soil is deficient in coarse material and has appreciable plasticity, admixtures of coarse, granular materials, such as sand, slag, gravel, or crushed stone, may prove beneficial. It has been observed that the efficiency of rounded gravel may be increased considerably by either crushing or adding angular fragments to the rounded material.

If the active portion of the soil mortar, because of domination of clay, is high in both plasticity and shrinkage properties, admixtures of either porous silt or materials such as hydrated lime, diatoms, etc., having high shrinkage limits may serve to reduce the shrinkage properties without reducing too much the plasticity of the soil. Penetrative bituminous materials applied to reduce the moisture capacity of the clay may prove beneficial.

TABLE 8.—*Test results on several soils combined with various admixtures*

Soil No.	Admixture	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent		Volumetric change
				Limit	Ratio	Centrifuge	Field	
		P. ct.	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*
3,770	None	24	0	20	1.7	15	24	
	60 per cent A sand [1]	25	0			10	25	
	20 per cent mica	44	0	38	1.2	23	44	
	15 per cent peat	37	0	28	1.3	26	34	
	15 per cent diatoms	73	38	36	1.3	38	42	
	6 per cent colloids	39	20	16	1.6	32	28	
4,056	None	66	40	11	2.1	[2] 53	29	
	60 per cent A sand	36	20	17	1.9	23	27	
	20 per cent mica	70	42	16	1.8	63	51	
	15 per cent peat	72	38	15	1.8	66	47	
	15 per cent diatoms	89	54	28	1.6	75	52	
	6 per cent colloids	89	62	10	2.1	[2] 133	40	
5,041	None	65	36	14	1.9	55	50	68.4
	15 per cent hydrated lime	84	58	24	1.6	59	42	28.8

[1] Angular sand consisting of crushed diabase.
[2] Waterlogged.

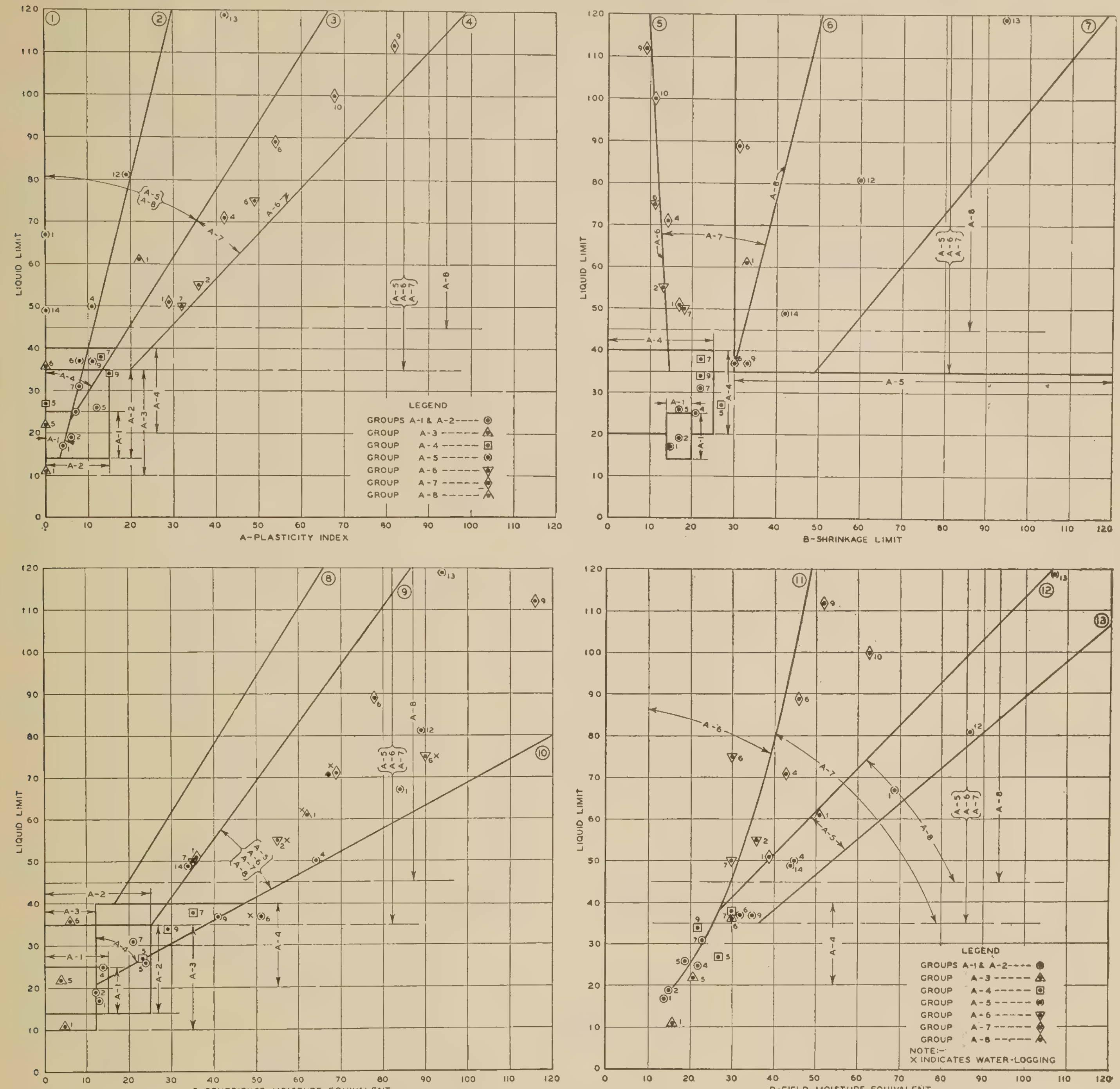


FIGURE 56.—THE SOIL IDENTIFICATION CHART

If the soil contains an abundance of coarse material and lacks clay and silt, cohesive materials obviously must be added. In this case either a proper clay binder may be added or the cohesionless coarse material may be treated with a highly penetrative bituminous material and covered with a light application of granular material.

If the soil, according to mechanical analysis, has proper grading but is low in both plasticity and shrinkage, and is to be placed on a very dry subgrade, the addition of a cohesive material may prove beneficial. Thus the gluey colloids which are detrimental to soil in large amounts may prove beneficial when present in very small amounts. Bentonite, for instance, added in the laboratory to a fine sandy loam in amounts not exceeding 3 per cent has the effect of introducing plasticity and resistance to erosion without increasing the shrinkage in detrimental amounts. Admixtures of bituminous materials, referred to above, may also serve this purpose very efficiently.

The data contained in Table 8 illustrate how the test constants disclose the influence exerted by different kinds of admixtures upon the characteristics of soils.

It is interesting to note that when mica and diatoms are added to the nonplastic soil, No. 3,770, both the shrinkage limits and the field moisture equivalents are very appreciably increased. The same is true when diatoms are added to the plastic soil, No. 4,056. When mica is added to the plastic soil only the field moisture equivalent is very appreciably increased.

It is interesting also to note that the volumetric change of soil No. 5,041 is 68.4 and that of the mixture of this soil and hydrated lime is only 28.8.

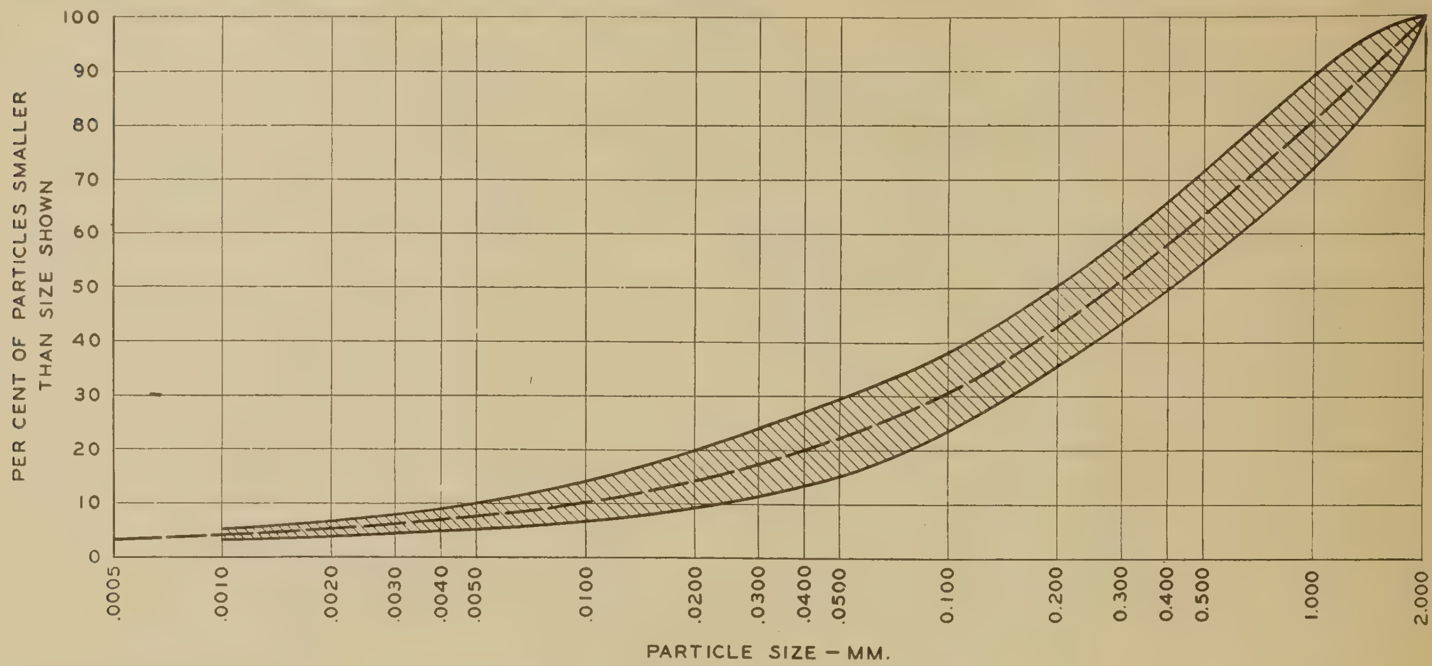


FIGURE 57.—GRADING OF GOOD SOIL MORTARS

GRADING AND CONSTANTS OF SUBGRADE GROUPS REVIEWED

The gradings of the various subgrade groups and the values of the test constants characteristic of them are repeated here as an aid to the discussion of soil identification, with which this part of the report is chiefly concerned.

Group A-1.—Grading: Material retained on the No. 10 sieve not more than about 50 per cent. The soil mortar, that fraction passing the No. 10 sieve, to consist of clay, 5 to 10 per cent; silt, 10 to 20 per cent; total sand, 70 to 85 per cent; and coarse sand, 45 to 60 per cent. Average effective size approximately 0.01 millimeters and uniformity coefficient greater than 15. The band shown in Figure 57 illustrates graphically the grading of good soil mortars.

Constants: Liquid limit not less than 14 nor greater than 25; plasticity index approximately equal to that indicated by curve 2 (fig. 56, A) and seldom greater than 8; shrinkage limit seldom less than 14 or greater than 20; and centrifuge moisture equivalent not apt to be greater than 15.

Fraction passing the No. 200 sieve—see constants of kaolin, p. 134, and Group A-5 subgrade below.

Group A-2.—Grading: Not less than about 55 per cent of sand in the soil mortar.

Constants: Liquid limit generally not less than 14 or greater than 35; a plasticity index of zero with a significant shrinkage limit or a plasticity index greater than zero and less than 15 with or without a significant shrinkage limit; centrifuge moisture equivalent not greater than 25.

Group A-3.—Grading: Effective size not likely to be less than 0.10 millimeters.

Constants: Liquid limit not appreciably greater than 35; no plasticity index; no significant shrinkage limit; centrifuge moisture equivalent less than 12.

Ability of sands to resist sliding when wet indicated as follows: Liquid limits of 10 to 14 signify beach and other rounded sands which slide easily; liquid limits of 30 to 35 indicate rough angular particles which do not slide easily. In addition, liquid limits when lower than field moisture equivalents indicate materials which flow under partial saturation; when equal to the field moisture equivalents, the liquid limits indicate average sands which flow under full hydrostatic uplift. Liquid limits greater than field moisture equivalents indicate rough-grained sands which flow only when in a state less consolidated than that represented by the field moisture equivalent. (See fig. 58.)

Group A-4.—Grading: Less than 55 per cent sand.

Constants: Liquid limit seldom less than 20 or greater than 40; plasticity index not greater than those indicated by curve 3; shrinkage limit not likely to be greater than 25; centrifuge moisture equivalent approaching those indicated by curve 10, between 12 and 50; when greater than liquid limit indicates varieties of soils inclined to be especially unstable in the presence of water; field moisture equivalent equal to or somewhat greater than those indicated by curve 11, with a maximum of about 30.

Increase in expansive properties generally indicated when shrinkage limits exceed 20 and approach those represented by curve 6; especially likely when field moisture equivalent exceeds centrifuge moisture equivalent.

Group A-5.—Grading: Less than 55 per cent sand. (Exceptions occur.)

Constants: Liquid limit usually greater than 35; plasticity index seldom greater than those indicated by curve 3; centrifuge moisture equivalent greater than 12, often lying between curves 9 and 10; not likely to water-log. (Exceptions occur.)

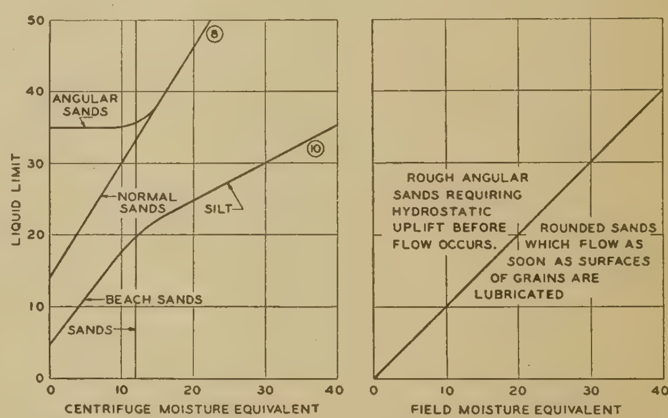


FIGURE 58.—SIGNIFICANT RELATIONS BETWEEN TEST CONSTANTS FOR GROUP A-3 SUBGRADE SOILS

Shrinkage limit generally greater than 30 and greater than 50 for very undesirable members of this group. May approach values indicated by curve 6 for silts containing peat and approach those indicated by curve 7 for soils containing either diatoms or mica in appreciable amount. Field moisture equivalent approaching those indicated by curve 12 for silts containing peat in appreciable amount and those indicated by curve 13 for highly elastic soils containing mica or diatoms in appreciable amount. The kaolins, representing good binders, are members of group possessing relatively high plasticity indices and low field moisture equivalents.

Group A-6.—Grading: Seldom contains less than 30 per cent clay.

Constants: Liquid limit usually greater than 35; plasticity index approximately represented by curve 4; shrinkage limit not likely to be appreciably greatr than that indicated by curve 5; centrifuge moisture equivalent test generally productive of water-logging; likely to lie between curves 9 and 10; field moisture equiva-

lent seldom exceeding those indicated by curve 11, but may be appreciably less for certain colloidal soils. Volumetric change generally greater than 17.

Group A-7.—Grading: Seldom contains less than 30 per cent clay.

Constants: Liquid limit usually greater than 35; plasticity index varies between those indicated by curves 3 and 4; shrinkage limit generally varies between those indicated by curves 5 and 6; centrifuge moisture equivalent varies between those indicated by curves 9 and 10; water-logging in centrifuge test may not occur even at very high moisture equivalents. Field moisture equivalent greater than those indicated by curve 11. Relatively low shrinkage limits with high field moisture equivalents indicate presence of colloidal organic matter. Relatively high shrinkage limits indicate the possibility of frost heave.

Group A-8.—Grading: Not significant.

Constants: Liquid limit greater than 45; plasticity index generally less than those indicated by curve 3; shrinkage limit indicated approximately by curve 6; centrifuge moisture equivalent between curves 9 and 10; field moisture equivalent likely to be greater than those indicated by curve 12.

Water-logging in the centrifuge test is characteristic of the mucks containing clay and colloids, whereas very high equivalents without water-logging are characteristic of peat not more than slightly decomposed.

NOMENCLATURE AND FORMULAS LISTED

The equations of the relationship curves 1 to 13, inclusive, and the basic formulas which have been developed in this report are listed below. The nomenclature for these equations is as follows:

$P.\ I.$ = plasticity index.
$L.\ L.$ = liquid limit.
$S.$ = shrinkage limit.
$R.$ = shrinkage ratio.
$C.\ M.\ E.$ = centrifuge moisture equivalent.
$F.\ M.\ E.$ = field moisture equivalent.
e = voids ratio.
e_o = voids ratio of dry sample.
V_v = volume of voids.
V_s = volume of soil particles.
V = volume of wet soil sample.
V_o = volume of dry soil sample.
C_o = volume change, percentage of V_o.
C_f = volumetric change, from $F.\ M.\ E.$, percentage of V_o.
$L.\ S.$ = lineal shrinkage, percentage of wet length.
M_w = weight of moisture.
W = weight of wet sample.
W_o = weight of dry sample.
w = moisture content, percentage of W_o.
w_v = moisture content, percentage of V_s.
G = specific gravity of soil particles.
P = porosity.
h = height of capillary rise in centimeters.
r = radius of capillary tube in centimeters.
a = width of pores in centimeters.
$S.\ F.$ = shrinkage force in grams per square centimeter.

EQUATIONS OF CURVES FOR SOIL IDENTIFICATION CHART

Curve 1.—$P.\ I. = 0$

Curve 2.—$P.\ I. = 0.25\ L.\ L.$

Curve 3.—$P.\ I. = \dfrac{L.\ L. - 14}{1.60}$

Curve 4.—$P.\ I. = \dfrac{L.\ L. - 14}{1.07}$

Curve 5.—$S = 21 - 1.1\sqrt{L.\ L. - \dfrac{L.\ L.^2}{800}}$

Curve 6.—$S = \dfrac{L.\ L. + 86}{4.1}$

Curve 7.—$S = \dfrac{L.\ L. + 26}{1.24}$

Curve 8.—$C.\ M.\ E. = \dfrac{L.\ L. - 14}{1.61}$

Curve 9.—$C.\ M.\ E. = 0.72\ L.\ L.$

Curve 10.—$C.\ M.\ E. = \dfrac{L.\ L. - 14}{0.55}$

Curve 11.—$F.\ M.\ E. = \sqrt{15.2\ (L.\ L. - 16.3)} + 9$

Curve 12.—$F.\ M.\ E. = \dfrac{L.\ L. - 10}{1.03}$

Curve 13.—$F.\ M.\ E. = \dfrac{L.\ L. - 4}{0.85}$

BASIC SOIL FORMULAS

$$e = \frac{V_v}{V_s} \qquad (1)$$

$$= \frac{wG}{100} \qquad (10)$$

$$P = \frac{e}{1+e} \times 100 \qquad (11)$$

$$C_o = \frac{V - V_o}{V_o} \times 100 \qquad (2)$$

$$= \frac{e - e_o}{1 + e_o} \times 100$$

$$C_f = (F.\ M.\ E. - S)\ R \qquad (18)$$

$$= \frac{F.\ M.\ E. - S}{\dfrac{1}{G} + \dfrac{S}{100}} \qquad (19)$$

$$L.\ S. = \frac{1 - \sqrt[3]{\dfrac{100}{C_f + 100}}}{0.01} \qquad (20)$$

$$M_w = W - W_o \qquad (5)$$

$$w = \frac{M_w}{W_o} \times 100 \qquad (4)$$

$$w = \frac{W - W_o}{W_o} \times 100 \qquad (6)$$

$$V_s = \frac{W_o}{G} \qquad (7)$$

$$w_v = wG \qquad (8)$$

$$h = \frac{0.153}{r} \quad \text{(See reference (1).)}$$

$$S.\ F. = \frac{0.306}{a} \qquad (14)$$

$$S = w - \frac{V - V_o}{W_o} \times 100 \qquad (15)$$

$$= \frac{100\ G - 100\ R}{RG}$$

$$R = \frac{W_o}{V_o} \qquad (16)$$

$$= \frac{100\ G}{100 + SG}$$

SOIL SAMPLES ANALYZED BY GROUPS

In the pages which follow the test results from a large number of soil samples are tabulated and analyzed. With the aid of the soil identification chart, the place of each soil in the uniform subgrade classification is determined. From this procedure the reader may gain an idea of how identifications of this sort are accomplished in practice.

The plastic limit, shrinkage limit, centrifuge moisture equivalent, and field, moisture equivalent of a number of samples from each subgrade group are plotted as functions of the liquid limit on the soil identification chart, Figure 56, in order that the comparative relations of the constants may be studied in reference to the numbered curves 1 to 13.

Group A–1 and Group A–2 subgrades.—The constants and grading of samples of soil containing both coarse and fine materials are included in Tables 9 and 10, respectively.

Sample 1, which represents a stable soil from South Carolina, satisfies both the grading and the constant requirements of the Group A–1 subgrade.

TABLE 9.—*Group A–1 and Group A–2 subgrades, coarse fractions and binders. Constants of material passing the No. 40 sieve except as noted*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	17	4	15	1.9	13	14
2	19	6	17	1.8	12	15
3	20	7	18	1.8	13	16
4	25	7	21	1.7	14	22
5	26	12	17	1.8	24	19
6	25	5			23	23
7	31	8	22	1.6	21	23
8	25				19	27
9	28	13	22	1.7	16	21
10	30	15	21	1.7	20	22
9–B [1]	49	29			27	
10–B [1]	58	34			36	
9–C [2]	24				5	25
10–C [2]	23				2	27

[1] Fraction passing No. 200 sieve.
[2] Fraction passing No. 40 sieve and retained on No. 200 sieve.

TABLE 10.—*Mechanical analyses of Group A–1 and Group A–2 subgrades*

Sample No.	Gravel particles larger than 2 millimeters	Mortar				
		Coarse sand 2.0 to 0.25 millimeter	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Good soil mortar		[1] 45–60		10–20	5–10	3–5
1	10	51	25	16	8	6
2	1	37	33	18	12	5
3	1	21	58	6	15	12
4	3	26	45	16	13	9
5	66	31	29	18	22	15
6	0	1	87	5	7	5
7	8	36	30	26	8	3
8	0	45	29	16	10	4
9	50	37	39	3	21	18
10	48	53	19	5	23	20

[1] Total sand 70 to 85 per cent.

Sample 2, which was obtained from a soil performing satisfactorily as fill in Escambia Bay, Fla., satisfies the constants but not the grading requirement of a Group A–1 material.

Its low centrifuge moisture equivalent of 12 indicates that the slight excess of clay above that generally indicating A–1 mortars is not likely to prove detrimental; and since the coarse sand in which this soil is deficient functions primarily to furnish the hardness required in wearing surfaces, this soil would probably prove highly stable as a subgrade.

The same is true for both sample 3, which was taken from a soil serving as a road surface in South Carolina, and sample 4, which was taken from a soil moderately stable when used as an untreated road surface in Madison County, Va., and highly stable when covered with bituminous surface treatments. (See fig. 59.)

FIGURE 59.—SURFACE-TREATED ROAD OF GROUP A–2 MATERIAL IN MADISON COUNTY, VA.

Sample 5, obtained from gravel used as road surface in Oklahoma; sample 6, from soil found by W. H. Dumont, of the United States Bureau of Fisheries, to be very unstable when occurring as bottom in Deep Creek, Ga.; and sample 7, from soil productive of frost heave in New Hampshire, fail to satisfy either the grading or the subgrade constant requirements of the Group A–1 material.

Sample 8, representing the Florida limerocks, satisfies the grading but not the constant requirements of the Group A–1 subgrade.

The high clay content of sample 5 is disclosed by the correspondingly high plasticity index of 12 and the centrifuge moisture equivalent of 24. The high plasticity index might indicate a desirable property, but the relatively high centrifuge moisture equivalent suggests water retentive properties not productive of stability. This detrimental property is probably offset to some extent by the fact that the field moisture equivalent is appreciably less than the centrifuge moisture equivalent.

The high silt content of sample 7 has produced a relatively high centrifuge moisture equivalent without raising the plasticity index in corresponding amount, one of the characteristics of soils productive of frost heave.

Special attention is called to sample 6, which, when submerged in water, has the properties of quicksand. This fact is disclosed by the nonuniform grading, which shows that the sample consists of 87 per cent fine sand, and the relatively high water-retentive property. The centrifuge moisture equivalent of this sample, containing but 12 per cent of particles smaller than 0.05 millimeter in diameter is practically the same as that of sample 5, which contains 40 per cent of particles smaller than 0.05 millimeter in diameter, and, furthermore, equals the field moisture equivalent which, in this case, can be assumed to indicate the amount of water required to fill the pores of the soil completely.

Sample 8 represents the nonplastic variety of the shell rocks. Tests performed on small beams of this material disclosed that when thoroughly dry this variety of limerock has practically no cohesion. Therefore, the stability of this material when used as a road surface must be due to the cohesion furnished by capillary pressure. Figure 60 shows how limerock occurs in nature.

Figure 60.—Examples of Shell Rock in Natural Location: A, Chalky Limerock of the Ocala, Fla., Region. B, Cochina Rock Near Ormond Beach, Fla. C, Ogus Rock, Lower East Coast of Florida

ALTERNATE METHOD OF INVESTIGATING GRADED MATERIALS DISCUSSED

A somewhat elaborate method suggested for investigating the properties of mixed materials consists of testing the mixture as such and also testing separately both the nonexpansive and expansive soils of which it is composed, and thus investigating soil mixtures in a manner similar to that employed when investigating bituminous mixes, concrete, or other materials consisting of a binder and an aggregate.

A procedure of this kind includes (*a*) the determination of the relative resistance furnished by the soil mortar to water absorption by means of slaking tests; (*b*) the determination of the relative strength of the mortar when dried, by means of a crushing or impact test; (*c*) the determination of the grading of the soil mortar by means of the combined sieve and hydrometer method; (*d*) the determination of both the plastic and shrinkage properties of that fraction of the material passing the No. 200 (0.074 millimeter) sieve by means of the test constants; and (*e*) the determination of the character of that fraction of the sample retained on the No. 200 sieve according to the procedure employed in identifying the characteristics of Group A–3 subgrades discussed subsequently.

Samples 9 and 10, Tables 9 and 10, representing, respectively, a stable and an unstable soil in South Carolina, serve to illustrate how coarse and fine materials may be investigated separately.

Table 10, for instance, discloses that sample 9, compared with the average good soil, contains an excess of clay and fine sand and is deficient in silt and coarse sand. Sample 10, compared with sample 9, contains a greater amount of coarse and a less amount of fine sand.

The binders of the two samples, 9–B and 10–B, Table 9, are similar in character, although the constants of 10–B are higher than those of 9–B. Both, however, have plasticity indices greater than that of kaolin (greater than that indicated by curve 3), combined with relatively low water retentive properties (curve 9). This indicates open-structure clays unlikely to shrink or expand in appreciable amount.

The constants of the sand fractions 9–C and 10–C, Table 9, are not radically different. Sample 9–C, however, which contains the greater amount of fine sand, has the higher centrifuge moisture equivalent. The liquid limit of sample 9–C is practically equal to the amount of water (field moisture equivalent) required to saturate the sample completely, whereas the liquid limit of sample 10–C is reached at a moisture content slightly below that required to saturate the soil completely.

From these evidences it would seem that the excellent quality of the binder primarily accounts for the fact that the soil represented by sample 9 can contain so large a clay content and yet remain stable.

That the same high clay content proved detrimental to the soil represented by sample 10 may be explained by the fact that, because of difference in grading, the surface area of the grains in the sand sample 9 may exceed by more than 40 per cent that of the grains in the sand fraction sample 10. Therefore, if the colloidal material, approximately the same in both samples, can be considered as a glue which coats the surfaces of the sand grains, sample 9, with the larger surface area, will cause the glue to be distributed in films thinner than those in sample 10. For this reason the quantity of glue (colloidal material) which causes the films to be excessively thick in sample 10 may not prove detrimental to sample 9.

Group A–3 subgrades.—The samples included in Tables 11 and 12 satisfy in both grading and constants the requirements of the cohesionless sand subgrades which provide good drainage and which do not have the tendency to heave under frost action.

Table 11.—*Group A–3 subgrades; Constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	11	0			5	16
2	20	0			4	25
3	19	0			3	17
4	20	0			3	21
5	22	0			4	21
6	36	0			6	30

Table 12.—*Mechanical analyses of Group A–3 subgrades*

Sample No.	Gravel particles larger than 2.0 milimeters	Coarse sand 2.0 to 0.25 milimeter	Fine sand 0.25 to 0.05 milimeter	Silt 0.05 to 0.005 milimeter	Clay below 0.005 milimiter	Colloids below 0.001 milimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	74	18	3	5	3
3	0	25	72	1	2	1
4	0	20	78	1	1	0
5	0	41	56	2	1	0

Figure 61.—Three-inch Grouted Brick Road Constructed about 1916 on Group A–3 Subgrade in Florida. Still in Service. Concrete Shoulders Constructed in 1921

Sample 1 was obtained from a Florida sand which serves excellently as subgrade, when prevented from flowing laterally, for relatively thin road surfaces. (See figs. 61 and 62.) Sample 2 represents that fraction of Potomac River sand passing the No. 20 and retained on the No. 100 sieve. Sample 3 was obtained from a California sand which serves excellently as a subgrade for concrete pavements 4½ inches thick (*23*). Sample 4 was taken from a Minnesota sand which becomes highly stable when treated with bituminous materials possessing penetrative properties in high degree, and covered with a thin application of granular material. Sample 5 is a New Hampshire sand which furnishes excellent support when treated in a manner similar to that described for sample 4. Sample 6 represents that fraction of crushed diabase passing the No. 20 and retained on the No. 100 sieve.

According to the significance of the relation existing between the liquid limits and the centrifuge moisture equivalents of these soils, the soils represented by sample 1 would be expected to have very low stability and those represented by sample 6 would be expected to have very high stability in the presence of water.

Additional information on the character of these soils is furnished by the relation between the liquid limit and the field moisture equivalent. Assuming that the field moisture equivalents of sands equal the amounts of water required to fill the pores of the sands completely when compressed by a very small but constant pressure, the glacial sands represented by samples 4 and 5, Table 11, are likely to flow when completely saturated (full hydrostatic uplift). The beach and river sands of samples 1 and 2 will flow when the grains are lubricated, rather than as a result of full hydrostatic uplift. The angular fragments of sample 6 require water in amounts greater than the field moisture equivalent to cause flow.

Figure 62.—Surface Treated Limerock Road Six Inches Thick in Florida

The fact that the liquid limit is greater than the field moisture equivalent indicates that, in order to flow, the angular particles must exist in a state looser than that represented by the field moisture equivalent. It appears from these facts that sample 1, when in the presence of water, is likely to be the least, and sample 6 the most stable of the sands listed in Tables 11 and 12.

Group A–4, subgrades.—Samples 1 to 9, inclusive, Tables 13 and 14, have constants indicative of those soils which have the tendency to heave under frost action and to lose stability as a result of water absorption even when not manipulated.

Samples 1 and 2 were made up, respectively, from commercial rotten stone and chalk. Although these materials are not natural soils they have the properties common to the A–4 subgrades. Sample 3 was composed of marl from Florida. Marl when kept dry and not subjected to frost action serves excellently as base course material. Samples 4 and 5 were obtained from New Hampshire silts which have been observed to heave in detrimental amounts under frost action. Sample 6 was obtained from a silt found in Missouri. Pavements laid on this soil have been observed to crack in appreciable amount. Samples 7 and 8 represent soils in the Minnesota frost-boil area which heave under frost and lose stability during the spring thaws. (See fig. 63.) Sample 9 was composed of lithopone, which has been found by Prof. Stephen Taber to suffer important frost heave.

In grading this soil should belong to the highly colloidal clays of either Group A–6 or A–7. The plasticity index of 15 compared with the liquid limit of 34 is slightly higher than those designating the A–4 subgrades. The relatively high shrinkage limit and low field moisture equivalent, indicating that this material is unlikely to shrink in appreciable amount, suggests the performance of silt instead of clay. Com-

TABLE 13.—*Group A–4 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	32	11	22	1.7	[1] 39	24
2	29	6	28	1.5	34	26
3	38	12	24	1.6	37	30
4	25	0	25	1.6	19	24
5	27	0	27	1.6	23	27
6	32	10	24	1.7	29	28
7	38	13	22	1.6	35	30
8	32	14	22	1.7	33	27
9	34	15	22	2.0	29	22

[1] Water-logged.

TABLE 14.—*Mechanical analyses of Group A–4 subgrades*

Sample No.	Gravel particles larger than 2.0 millimeters	Coarse sand 2.0 to 0.25 millimeters	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	0	7	57	36	17
2	0	0	13	37	50	3
4	1	14	34	28	24	1
5	0	13	21	42	24	2
6	0	5	10	58	27	12
7	1	7	20	52	21	4
8	1	9	26	46	19	4
9	0	0	2	3	95	67

parison of the constants of this material with those of statistical soils also discloses its inactivity.

A statistical soil, for instance, containing 95 per cent clay has a liquid limit of 132, a plasticity index of 74, a shrinkage limit of 10, a centrifuge moisture equivalent of 95, and a field moisture equivalent of 51. Thus, lithopone, irrespective of its grading, has relatively very low cohesive properties, and this fact, combined with its negligible shrinkage properties and a liquid limit smaller than 40, causes lithopone to be grouped with the silts.

Group A–5 subgrades.—The constants and the grading characteristic of Group A–5 material are shown in Tables 15 and 16. In grading, these samples contain either clay or silt in dominating amounts. Only one sample has a liquid limit smaller than 35; all samples have plasticity indices smaller than those indicated by curve 3 and all but two samples have shrinkage limits equal to or greater than 30. The four samples whose shrinkage limits are not given have values of this quantity higher than the liquid limit.

Samples 1, 2, and 3 were made, respectively, from commercial hydrated lime, pumice, and talc. Samples 4 and 5 were made from barium sulphate and reground quartz, respectively. Professor Taber found the former to heave very appreciably and the latter to heave but very little as a result of frost, in experiments performed in the laboratory. Samples 6 and 7 were composed of silt found in St. Peter, Minn., which, according to F. C. Lang, heaved several feet during the winter of 1928-29. Samples 8 and 9 were obtained from New Hampshire silts which have been found by W. F. Purrington to heave in appreciable amount under frost action. Samples 10 and 11 were made up from Oregon silts which have been observed by R. H. Baldock to suffer important frost heave.

In grading, samples 6, 7, and 8, which are comparable to barium sulphate (sample 4), contain clay in dominating amount. Samples 9, 10, and 11, which are comparable to reground quartz (sample 5), contain silt in dominating amounts. Sample 9 contains mica, while samples 10 and 11 contain an appreciable amount of organic matter.

The soils represented by samples 6 to 11, inclusive, in addition to appreciable frost heaving, are likely to lose stability during thaws. In addition, these soils are capable of raising water in detrimental amounts through great heights during frost action.

Samples 12 and 13 were obtained from Maryland soils containing diatoms in appreciable amount. Soils of this character, especially when their shrinkage limits are equal to or greater than 50, are almost sure to produce pavement failure because of their high porosity.

FIGURE 63.—TYPICAL FROST BOIL CAUSED BY LACK OF STABILITY IN SOIL

TABLE 15.—*Group A–5 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	67	0			84	69
2	58	0			36	62
3	43	17	41	1.3	48	36
4	50	11			[1] 64	45
5	47	0			22	28
6	37	8	30	1.5	[2] 51	32
7	36	9	28	1.5	[2] 40	31
8	45	18	37	1.4	54	37
9	37	11	33	1.5	41	35
10	40	8	33	1.3	36	37
11	43	9	37	1.3	38	41
12	81	19	60	0.9	89	87
13	119	42	94	0.7	94	107
14	49	0	42	1.2	34	44
15	34	12	26	1.5	25	30

[1] Tendency to waterlog. [2] Waterlogged.

TABLE 16.—*Mechanical analyses of Group A–5 subgrades*

Sample No.	Gravel particles larger than 2.0 millimeter	Coarse sand 2.0 to 0.25 millimeter	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	1	7	86	(1)	(1)
2	0	0	10	78	12	8
3	0	0	9	74	17	7
4	0	0	1	6	93	26
5	0	0	8	78	14	4
6	0	2	5	16	77	17
7	0	0	14	27	59	40
8	0	1	6	32	61	8
9	0	1	5	60	24	2
10	3	6	23	57	14	0
11	3	6	27	60	7	0
12	0	0	29	42	29	12
13	0	0	40	28	32	9

[1] Flocculated.

With a specific gravity of 2.0, for instance, and a shrinkage limit equal to 50, these soils in their dried state have voids in amount equal to the volume occupied by the soil particles. Figure 64 shows a dried pat of sample 13 floating on the surface of water.

Samples 14 and 15 were taken from micaceous soils in Maryland. These samples are representative of the elastic subgrades which are productive of the early cracking in concrete pavements illustrated in Figure 15, A, Part I, of this report.

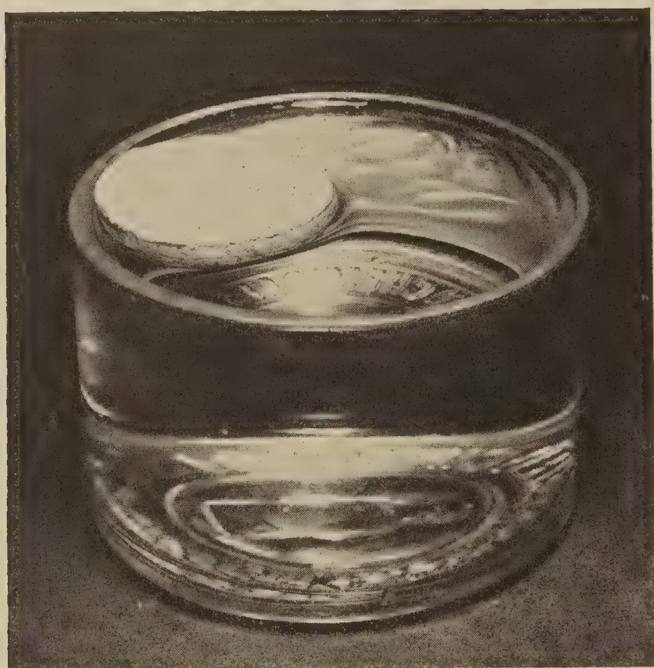


FIGURE 64.—DRIED PAT OF DIATOMACEOUS EARTH (SAMPLE 13, TABLE 15) FLOATING IN WATER AFTER BEING COATED WITH SHELLAC. APPARENT SPECIFIC GRAVITY LESS THAN ONE

A comparison of Tables 14 and 16 shows that it would be impossible to distinguish Group A–4 from Group A–5 subgrades on the basis of the mechanical analyses alone. The difference in the constants (Tables 13 and 15), however, indicates a difference in the characteristics of the soils. The higher liquid limits combined with the high shrinkage limits and field moisture equivalents differentiate those soils possessing elasticity (Group A–5) from those which are compressible (Group A–4).

Samples 4 and 7, according to the mechanical analyses, contain colloids in amount sufficient to produce high plasticity and shrinkage properties in appreciable amount, were the colloids active. These soils, however, according to their constants shown in Table 15, have low plasticity and negligible shrinkage properties and, consequently, should be grouped with the A–5 instead of the A–7 subgrades.

It is interesting to note that two soils (samples 7 and 9) which are likely to heave in appreciable amount have similar constants in spite of the fact that sample 7 contains 40 per cent of particles of colloidal size and 59 per cent of clay, whereas sample 9 contains but 2 per cent of colloidal particles and 24 per cent of clay. This is further evidence that grain size alone does not control the performance of subgrade soils.

Group A–6 subgrades.—The samples included in Tables 17 and 18 illustrate the constants and the grading characteristic of the nonelastic colloidal clay subgrades. Sample 1 was obtained from a colloidal clay soil which has proved troublesome because of sliding in a cut in Missouri, and sample 2 from a colloidal clay soil which has caused similar difficulties when used for fill material, in the same State. Samples 3 and 4 were composed of colloidal clays furnished by a survey of the soil existing under the Potomac River at Washington, D. C. These soils were considered unfit for use as hydraulic fill material. Sample 5 represents a very highly colloidal clay soil productive of landslides in Virginia. This soil contains about 30 per cent of material so fine that it remains in suspension for weeks. If located on an impervious undersoil, this soil when in a soft condition acts as a lubricant facilitating the sliding of the upper soil layers. (See Fig. 65.) Sample 6 was obtained from a colloidal soil from Texas, and sample 7, when occurring as subgrade, produced failure in a limerock base course being constructed on a road north of Gainesville, Fla.

TABLE 17.—*Group A–6 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	51	31	13	1.9	[1] 56	34
2	55	36	13	1.9	[1] 55	36
3	58	40	12	1.9	[1] 45	28
4	57	40	12	1.9	[1] 45	28
5	132	101	11	1.9	[1] 178	33
6	75	49	11	2.0	[1] 90	30
7	50	32	18	1.7	35	30

[1] Waterlogged.

TABLE 18.—*Mechanical analyses of Group A–6 subgrades*

Sample No.	Gravel-particles larger than 2.0 millimeters	Coarse sand 2.0 to 0.25 millimeters	Fine sand 0.25 to 0.05 millimeters	Silt 0.05 to 0.005 millimeters	Clay below 0.005 millimeters	Colloids below 0.001 millimeters
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	0	1	52	40	15
2	0	0	11	57	32	16
3	0	19	30	11	50	24
4	16	25	12	30	43	27
5	0	3	3	6	88	78
6	0	3	20	29	48	24
7	0	28	32	17	23	10

The relatively large plasticity indices and the relatively small shrinkage limits and field moisture equivalents, combined with waterlogging in the centrifuge test, identify samples 1 to 6, inclusive, as containing clay highly active in character. This is emphasized by the mechanical analyses, which disclose that only one of the six samples contain more than 50 per cent of clay.

Sample 7, on the basis of its grading and its shrinkage limit would be classed an A–2 subgrade. Its volumetric change of 20.4 ($(30-18)\times1.7$) and its plasticity index of 32 for a combined clay and silt content not larger than 40 definitely place this soil with the colloidal clays. The clay contained in this sample was similar in stickiness to chewing gum. This clay is the only subgrade material found thus far in the subgrade investigations which has an activity practically equal to that of bentonite. It is interesting to note that the plasticity index of this sample, which contains but 23 per cent of clay, is more than double that of lithopone, which contains 95 per cent of clay.

The presence of gluey colloids generally serves to protect the Group A–6 subgrades from detrimental frost heaving. These soils are likely to contain relatively large amounts of the unfreezable water productive of frost heave. The rate at which capillary moisture moves through them, however, is so slow that detrimental heave will probably not occur unless well-defined seepage planes furnish the necessary moisture at the required rate.

Group A–7 subgrades.—The samples contained in Tables 19 and 20 illustrate the grading and the constants characteristic of those clay subgrades which are likely to be elastic under certain prevalent conditions. Sample 1, furnished by F. V. Reagel, was taken from an expansive clay on which a concrete pavement in Missouri cracked in appreciable amount during the setting period of the concrete. Sample 2, furnished by W. C. McKnown, was obtained from an expansive clay in Kansas on which similar cracking occurred in a concrete pavement. Sample 3, furnished by W. D. Ross, was obtained from an expansive clay in Colorado on which similar cracking occurred in the pavement.

TABLE 19.—*Group A–7 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	51	29	17	1.7	36	39
2	39	20	15	1.9	28	27
3	42	24	14	1.9	35	34
4	71	42	14	1.8	[1] 69	43
5	63	38	17	1.8	[1] 60	46
6	89	54	31	1.9	[2] 78	46
7	67	37	20	1.7	[2] 46	39
8	68	42	12	2.0	[1] 67	47
9	112	82	9	2.0	116	52
10	100	68	11	2.0	[1] 132	63
11	83	53	13	1.9	[1] 72	64

[1] Waterlogged. [2] Tendency to waterlog.

TABLE 20.—*Mechanical analyses of Group A–7 subgrades*

Sample No.	Gravel particles larger than 2.0 millimeters	Coarse sand 2.0 to 0.25 millimeters	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	0	7	55	38	24
2	0	1	14	52	33	16
3	0	2	5	45	48	21
4	0	2	9	43	46	22
5	0	1	13	43	43	24
6	0	0	2	6	92	78
7	0	0	5	23	72	29
8	0	3	8	35	54	27
9	0	0	4	15	81	18
10	0	1	9	54	36	16
11	0	1	14	28	57	21

These three soils, it will be noted, are similar in several respects. They all contain approximately 50 per cent of silt; the plasticity indices of the three soils bear similar relationships to their liquid limits, being slightly less than those represented by curve 4; the centrifuge moisture equivalents of all three soils are relatively small, being approximately equal to those represented by curve 9; and the field moisture equivalents are either approximately equal to or greater than the centrifuge moisture equivalents. It will be recalled that the micaceous silts of the Group A–5 subgrade, on which cracking occurred during the early age of the concrete, also have field moisture equivalents generally larger than the centrifuge moisture equivalents.

Samples 4 and 5 were composed of gumbos from the Red River Valley, Minn., which do not suffer detrimental frost heave and which are stabilized when oil-treated and covered with granular material.

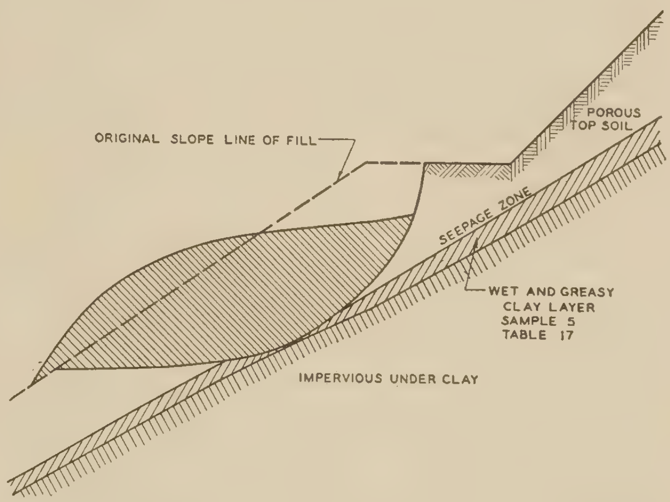


FIGURE 65.—DIAGRAM ILLUSTRATING LANDSLIDE CAUSED BY PRESENCE OF HIGHLY COLLOIDAL CLAY SUCH AS SAMPLE 5, TABLE 17

Sample 6 was composed of kadox, which, according to laboratory experiments performed by Professor Taber (*24*), is productive of important frost heave. Except for low colloidal activity and high shrinkage limit this material seems to be a normal A–7 subgrade. The shrinkage limit of 31, however, is approximately equal to those of the Group A–5 silts of New Hampshire (samples 8 and 9, Table 15) in which detrimental heave due to frost is likely to occur.

The relative activity of the colloids in the two samples, 5 and 6, is disclosed when their constants are expressed as decimals of those characteristic of the average or statistical soils. Thus the plasticity index of sample 5 is 1.31 times that of an "average" soil containing 43 per cent of clay, and the plasticity index of sample 6 is only 0.76 times that of an "average" soil containing 92 per cent clay. Kadox, therefore, possesses both the relatively inactive colloids and the high porosity productive of frost heave.

Sample 7, obtained from a soil in Michigan similar to sample 6, has a relatively low plasticity index, only 0.69 times that of an "average" soil containing 72 per cent clay; and also a relatively high shrinkage limit, 1.82 times that possessed by the average soil referred to. This soil has been observed to have detrimental elasticity in dry weather. Increasing the moisture content of the soil, however, causes a decrease in its elasticity. This is true also for certain varieties of the micaceous silts.

This soil displays the properties indicative of frost heave. The current surveys, however, have not yet disclosed whether or not such heave occurs.

Sample 8 was obtained from a soil which became exceedingly troublesome because of sliding when used in a high fill in Missouri. The organic matter which causes this soil to be placed in the A–7 group has decomposed to such an extent that its presence is disclosed only by the relatively large field moisture equivalent.

Sample 9 represents a flocculated, highly colloidal soil from Mississippi. Only the absence of water-logging with a centrifuge moisture equivalent as large as 116 prevents this soil from being grouped with the A–6 subgrades. This soil, which contains both lime and gypsum, supports concrete slabs which have warped in detrimental amounts. Soil of this character can not be used efficiently in fills and in cuts it should be separated from concrete pavements by a good topsoil base course at least 2 feet thick.

Sample 10 was made up from a Texas black waxy soil. Its high field moisture equivalent causes it to be grouped with the A–7 subgrades. Sample 11 is another soil of the gumbo type which proved troublesome when used in fills in Arkansas.

FIGURE 66.—DRY LAND FILL CONSTRUCTED ON GROUP A–8 SUBGRADE IN VIRGINIA

Group A–8 subgrades.—Constants of representatives of the soft peats and mucks are contained in Table 21. The grading of these materials is not significant.

TABLE 21.—*Group A–8 subgrades; Constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	61	22	33	1.4	[1] 62	51
2	62	26	38	1.4	[1] 56	47
3	59	19	31	1.4	68	51
4	265	0	141	0.5	263	265
5	445	0	187	0.3	395	440

[1] Waterlogged.

Samples 1 and 2 were obtained from Potomac River bottom muck which failed to support hydraulic fills without displacing laterally. The high shrinkage limits and high field moisture equivalents indicate the presence of partly decomposed organic matter. The relatively high plasticity indices and the water logging in the centrifuge test disclose the presence of clay or colloidal organic matter which in combination with the partly decomposed organic matter comprises muck. Sample 3 was obtained from Minnesota muck, which has a tendency to displace laterally when supporting fills. The presence of sand in this material may account for the absence of water logging in the centrifuge test. Sample 4 was composed of a peat which failed to support a dry land fill in Virginia (fig. 66) and sample 5 from a peat which has proved inadequate to support fills in Minnesota.

Samples 4 and 5 illustrate the very high water-absorptive properties characteristic of the peat soils which have not yet reached the colloidal state by decomposition. That these two soils have not yet reached this state is indicated by the fact that their plasticity indices are equal to zero and by the absence of water logging in the centrifuge test.

CONCLUSIONS SUMMARIZED

The foregoing discussion serves to emphasize that relatively few and comparatively simple laboratory tests may serve to identify fairly accurately the important characteristics of subgrade soils. Consequently these tests may serve to identify the dominating constituents composing the soils and to suggest the proper corrective measures to be used in pavement construction.

The following generalizations, based on the data given in the preceding pages, illustrate the service performed by the test constants in identifying the characteristics of subgrade soils.

Graded materials having centrifuge moisture equivalents greater than 15 have been found to be related to loss of subgrade stability in the presence of water.

Groups A–4, A–5, and A–7 subgrades having field moisture equivalents approximately equal to or greater than the centrifuge moisture equivalents have been found to be conducive to the cracking which occurs in pavements during the early life of the concrete.

Groups A–2, A–4, and A–5 subgrades having relatively high centrifuge moisture equivalents and Group A–7 subgrades having exceptionally high shrinkage limits seem to favor detrimental frost heave.

Group A–3 subgrades and groups A–6 and A–7 subgrades having relatively high plasticity indices are unlikely to heave under frost action.

Groups A–6 and A–7 subgrades are likely to shrink and expand in appreciable amount.

Group A–7 subgrades, having relatively high shrinkage limits, and Group A–5 subgrades, because of their elasticity, are likely to prove troublesome in the preparation of the subgrade.

The plastic varieties of the Group A–5 subgrades, with exceptionally low field moisture equivalents, generally have the properties required in good binders for sand-clay and topsoil roads.

The principal purpose of this report is to describe a method according to which soil research yielding profitable results may be performed. It should serve also to illustrate (*a*) the effort which may be required to interpret the test constants properly, (*b*) the character of the information which may be obtained by the intelligent use of the test constants, (*c*) the necessity for understanding the full significance of each constant, and (*d*) the impossibility of stating in simple terms the general procedures by means of which the constants of all soils may be readily interpreted.

It is expected that future invetigations will improve the procedure for making soil tests, and will increase the precision and facility with which soils may be identified by the use of test constants.

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THE SOIL PROFILE AND THE SUBGRADE SURVEY [a]

Reported by W. I. WATKINS, Assistant Soil Surveyor, Bureau of Chemistry and Soils, and HENRY AARON, Assistant Highway Engineer, Bureau of Public Roads

SUBGRADE soil profiles, the importance of which with respect to road construction is discussed in this report, are studied and mapped in the field according to procedures suggested by the pedologist. However, the highway engineer is interested only in those soil properties upon which the serviceability of pavements depend. Some of these properties are often of minor significance in an agricultural sense. In other cases the scale adopted in the publication of reports of agricultural soil surveys is too small to show all of them. For this reason attention in field and laboratory is directed specifically to those properties which are important in road construction. The final map of the subgrade soil profile discloses those engineering properties of soils which are determined by laboratory tests, as well as those physical characteristics which are identified by the soil scientist.

Information furnished by the soil scientist which assists the engineer both in the study of subgrade profiles and in the interpretation of existing soil data is contained in the reports listed in the attached bibliography.[1]

THE SOIL PROFILE DESCRIBED

According to Dr. C. F. Marbut (*1*) the soils of the United States may be divided into two large categories: (1) Pedalfers and (2) pedocals. The pedalfers are soils that tend to accumulate iron and aluminum and have no horizon[2] of lime carbonate accumulation, even if the soils have limestone as parent material. The accumulation of lime carbonate and other salts is a characteristic feature of pedocals. This discussion is based on studies covering a wide range of pedalfer and a few pedocal soil types established by the Bureau of Chemistry and Soils.

A vertical section of the weathered soil layers, having different characteristics though uniformly weathered within themselves, is known as the soil profile. In soil technology the term "soil profile" applies to the weathered layers which constitute what is known as the solum and the material immediately underlying it. In the well-drained upland soils of the humid region the solum is ordinarily less than 10 feet in thickness and is usually composed of a lighter-textured upper and a heavier-textured lower layer, both of which may have sublayers.

A soil type is the soil unit and is as definite an object as any other classified object, animal, or plant, and always presents approximately the same physical characteristics (*5, 6*). A soil series is a group of soil types that have profiles uniform in all respects except the texture of the surface layer.

Practically all modern soil research is conducted upon the basis of the soil-type profile. There is no desire to create the impression that every soil type or soil layer differs widely from all other soil types or soil layers; but it is believed that the method of classification based on soil types furnishes the most economical basis for studying soils and utilizing the information obtained for any given purpose. After the properties or behavior of the layers of a soil type are once determined the data obtained will be immediately and continuously available and may be of use in connection with studying any branch of science in any way pertaining to soil, such as agriculture, erosion, corrosion, drainage, road construction, and so on. Such information is applicable wherever the soil type from which the information was obtained is found. The behavior of soil layers common to the individual soil types of a soil series or of a closely related soil series may thus be ascertained by determining the behavior of such a common layer in one or more of the closely related soil types. Additional discussion of the soil profile is contained in a report by Joffe (*2*). Figures 1 to 4, inclusive, illustrate the character of the material encountered in the determination of soil profile.

SOIL PROFILE AS RELATED TO ROAD CONSTRUCTION

One can readily understand that the characteristics of the soil profile control percolation, capillarity, seepage—in fact, all internal water movement except that influenced by nonsoil agencies, such as animal burrows, root holes, etc. The influence of the soil layers on the nature of water movement will change if the soil is in any way disturbed. The degree of change will depend on the extent to which the natural characteristics are destroyed. The resistance to change of the natural characteristics and the subsequent effect on soil behavior are governed largely by the relative amounts of silt and colloids, and the chemical composition of the colloids (*7*). Certain other constituents such as diatoms and mica flakes have a decided influence upon the physical behavior of a soil (*8*). The soil characteristics, their descriptive terms, and the influence they may have on subgrade soil behavior are summarized in the following paragraphs:

Texture.—For sand or sandy loams: Coarse, medium, fine, very fine. Other soils classified by texture as loam, silt loam, clay loam, silty clay loam, clay. May reflect plasticity, density, slumping, porosity, expansion, contraction, capillarity, susceptibility to compaction, elasticity, percolation, water absorption, erosion, structure.

Color.—Black, red, brown, yellow, drab, gray, mottled. May reflect organic content, oxidation, leaching, chemical content, structure, erosive qualities.

Structure.—Buckshot, granular, columnar, cloddy, crumb, adobe, dense, massive, laminated, nut, mealy, structureless. May reflect capillarity, percolation, porosity, water absorption, internal drainage, bulking on being disturbed, susceptibility to compaction, elasticity.

Consistency.—Brittle, hard, compact, tough, tenacious, sticky, plastic, cheesy, friable, mellow, loose. May reflect capillarity, percolation, porosity, water absorption, internal drainage, bulking, compactibility, elasticity, supporting power.

Compactness.—Clay pan, hardpan; soils also classified by degree of compaction. May give idea of bulking, internal water movement, degree of support.

[a] Reprinted from PUBLIC ROADS, vol. 12, No. 7, September, 1931.

[1] Italic figures in parentheses refer to the reports listed in the bibliography on page 58.

[2] In standard soil descriptions for humid regions the light-textured surface layer is designated the A horizon, the underlying heavier-textured layer as the B horizon, and the third layer, consisting of the unweathered or incompletely weathered geological formation, as the C horizon. The A and B horizons constitute the real soil profile, the Solum horizon of Frosterus, while the C horizon is part of the parent geological formation not made significantly lighter or heavier in texture by soil-making processes. An horizon may have subhorizons such as A_1, A_2, etc. For highway purposes, the various layers are numbered consecutively from the surface down without designating any horizon. For example, instead of layers A_1, A_2, B, etc., we have layers 1, 2, 3, etc. Some soil surveyors designate these layers as zone 1, zone 2, zone 3, etc.

Cementation.—Classified by cementing material and degree of cementation. May give idea of bulking, internal water movement, degree of support.

Chemical composition.—Significant chemicals are iron, calcium, concretions, alkali, sesquioxides, exchangeable bases, pH value. May affect structure, adhesiveness, friability, plasticity, drainage and water movement, formation of hardpans, expansion, contraction, water retention, heat of wetting.

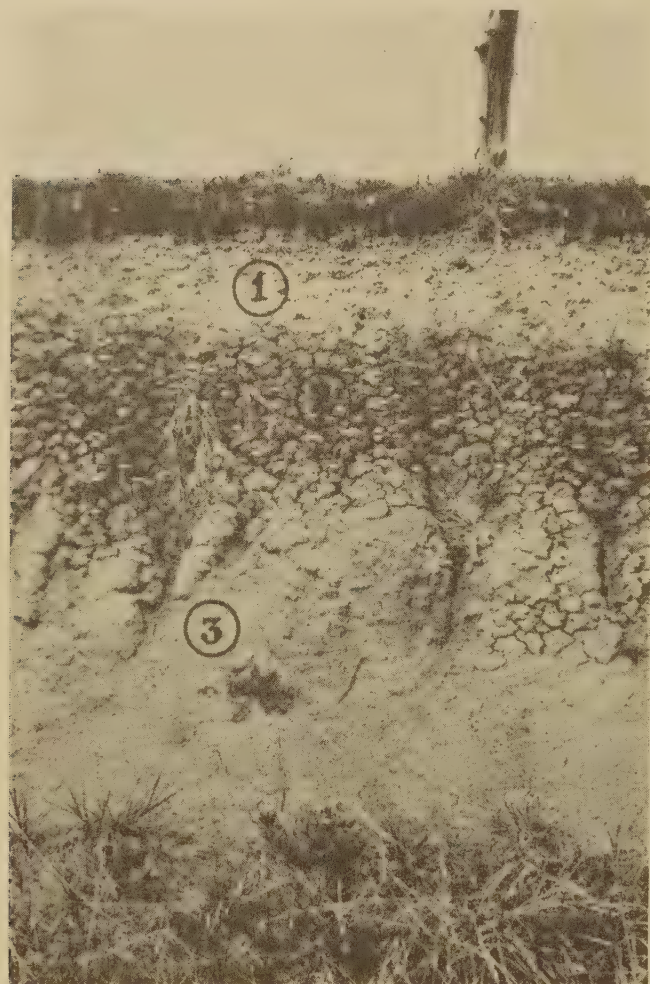

Figure 1.—Profile of Iredell Silt Loam Showing (1) Lighter-Textured Upper Soil Layer, (2) Heavy, Plastic, Sticky, Waxy Clay, and (3) Decomposed Rock

Organic content classified by kind of material and degree of decomposition; influences water absorption, temperature, firmness, stability, compactibility and springiness.

In nonalkali soils the percentage of colloids and their chemical composition are probably the most important factors controlling such soil characteristics as consistency, structure, and porosity. This is especially true of the silica sesquioxide ratio. Soils containing colloids with a low silica sesquioxide ratio, i. e., colloids high in the sesquioxides of iron and aluminum and low in the oxides of silica, shrink and expand but slightly at extremes of moisture content, are not very plastic or excessively sticky, and are more friable, more porous, and less plastic than soils containing an equal amount of colloids with a high silica sesquioxide ratio (*9*, *10*). Some highly colloidal clays of low sesquioxide ratio make better road subgrades than other soils containing less colloids with a high silica sesquioxide ratio (*11*).

Texture has usually been taken as an index of the rate of percolation, but structure is now considered of equal or greater importance (*12*), and the destruction of the natural structure may increase or decrease the rate of percolation. The variability in percolation rates of the various soil layers or soil material causes and controls seepage or lateral water movement. Seepage develops when the amount of water percolating through any one soil layer exceeds the percolating rate of that below. The amount of lateral movement is governed by the amount of water and differences in rate of percolation.

Figure 2.—Profile of Leonardtown Silt Loam, Showing (1) Grayish Brown Granular Silt Loam, (2) Brown, Granular Silty Clay Loam, (3) Gray, Compact, Sandy Hardpan, (4) The Friable Clay, and (5) The Compact Clay

It will be seen that water movement depends not only on the character of any particular layer but also on its relation to other associated layers and the effect of local climatic conditions. A silt loam may block or retard percolation in a soil profile where the silt is overlain by sands or may act as a water carrier if underlain by compacted sands or clays. In some soil profiles clay layers may act as water carriers if underlain by denser, less pervious layers. Stratified soils, dense clays, and hardpans are frequently causes of seepage. Silts, sands, and gravels are the most prolific carriers. Cleavage planes in geological materials are also channels for lateral water movement. Figure 5 shows cleavage planes in shale. The soil profile discloses these water-carrying layers in the field. Through its control of water movement, the soil profile furnishes a basis for the design of drains. The proper location of drains with respect to the soil profile is illustrated in Figure 6.

Figure 3.—Road Cut Through a Bed of Stratified Sands and Gravels in Glacial Drift. Notice How Gravel Deposit Ends Abruptly at Right of Photograph

The study of soil profiles has shown frost boils to be confined to certain soil materials or soil layers by themselves, or in combination with other soil materials or

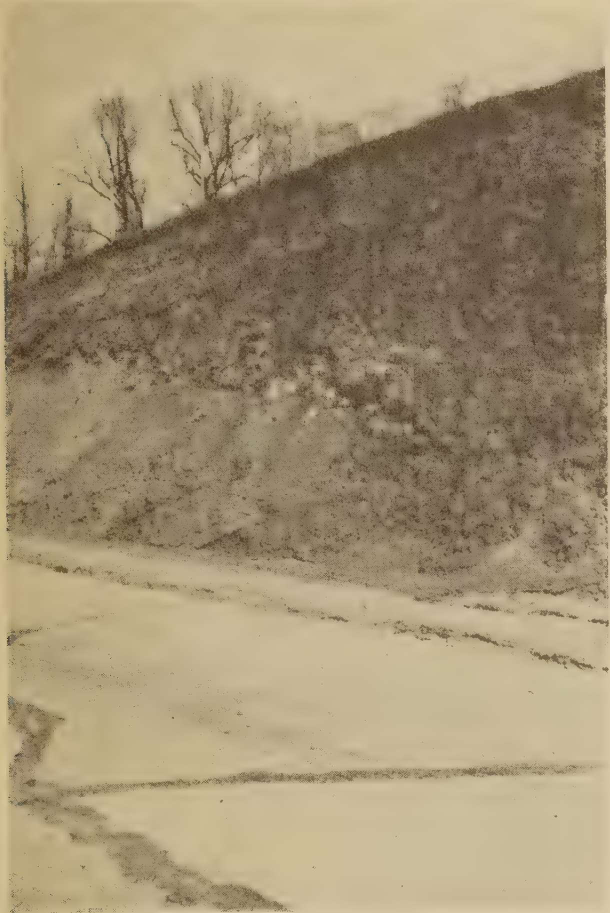

FIGURE 4.—A CUT THROUGH COLLINGTON LOAM. THE MATERIAL OUTLINED IN THE LOWER PART OF THE PHOTOGRAPH CONTAINS DIATOMS, IS POROUS AND LIGHT IN WEIGHT IN ITS NATURAL CONDITION, HOLDS ENORMOUS AMOUNTS OF WATER, HAS LITTLE STABILITY WHEN WET AND IS DISTINGUISHED BY A SOAPY FEEL

layers possessing certain definite characteristics. These studies have shown the very fine sands, silt loams, and silty clay loams having little or no apparent soil structure to be the greatest sources of frost boils where these soils or soil materials are found associated with water carriers or where they act as water carriers themselves. In New Hampshire, for example, frost boils were found to be associated mainly with stratified silty clay loams and very fine sandy loams that retarded percolation or had seepage characteristics, and carried the water into the subgrade, where it froze.

In Minnesota frost boils are found to occur most frequently in soil material having no readily determinable structure, and vary in degree of severity depending on the soil material and source of water. The more variable the soil and the more water available, the more severe the frost boil development. The extremely variable glaciated soil materials of this region probably develop the most severe heaving. Less severe heaving occurs in the structureless loessial soil material of southeastern Minnesota. Figure 7 is a diagrammatic illustration of a frost boil and its relation to the soil profile of a glaciated soil, while Figure 8 shows the same for a loessial soil. These drawings represent actual cases.

This structureless loessial soil material is a silt loam containing a high percentage of very fine sand, has a high water-holding capacity, is unstable when wet, and has a texture likely to produce maximum active capillarity. The material is underlain by a heavy, slightly pervious clay that restricts percolation and causes a water table to develop just above the clay. This water table may supply the water to the road surface by capillarity.

Capillarity is controlled by structure and density as well as by texture. The soil types overlying the loessial soil material seldom develop frost boils if in their natural condition, but if disturbed and used as fill in poorly drained areas are likely to become soft during the spring, as silty soils do not have resistant structure and are likely to have more active capillarity when the structure is destroyed.

FIGURE 5.—STRATIFICATIONS IN SHALE. THE CONTACT PLANES BETWEEN THE DIFFERENT SHALE STRATA ARE POTENTIAL PLANES OF WATER SEEPAGE

The soil profile offers a means for the study of the design and construction of fills and back slopes in different soil types and under different drainage conditions. With a knowledge of the physical characteristics of the various layers of the soil profile, the stability of the different layers in place or when combined in fill may be determined. This information will establish the slope of fills and back slopes which will

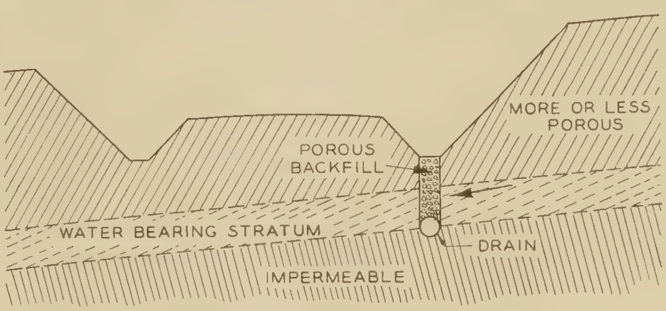

FIGURE 6.—DRAINS SHOULD EXTEND THROUGH THE ENTIRE DEPTH OF THE PERVIOUS LAYER AND INTO THE IMPERVIOUS LAYER

have to conform to the layer or material having the smallest angle of repose; or remedies will have to be adopted to increase its angle of repose. There is a possibility of establishing fairer grading contracts through a knowledge of the amount of the various soils or soil materials which are to be moved, as such data make possible a closer estimate of the cost of handling.

The characteristics of a soil and its value as a subgrade under different conditions are directly reflected in the condition of the road surface. Rigid pavements are affected by inequalities of subgrade support. Nonrigid pavements are chiefly affected by low road supporting power. Studies have shown that the subgrade

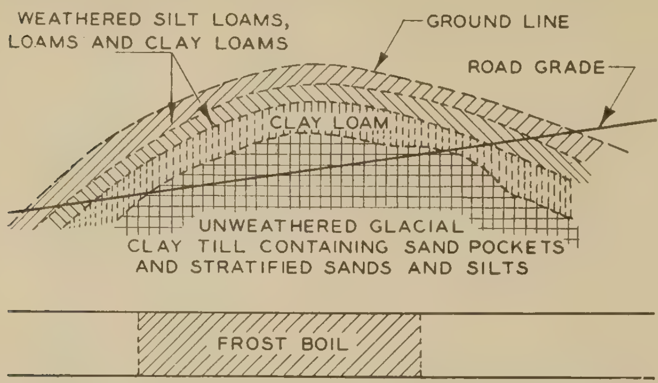

FIGURE 7.—FROST BOIL IN GLACIATED SOIL MATERIALS

soil exerts important influence upon the distance between transverse cracks in concrete roads, and that excessive longitudinal cracking develops on definite layers of certain soil types. Other soil characteristics, such as swelling, affinity for water, rebound, etc., are detrimental to concrete before it sets as it is then a flexible pavement. Figures 9 and 10 show the relation which exists between the various soil zones of a soil type and the pavement condition. In addition to the effect of the soil itself, these results are affected by climate, rainfall and temperature range. With a knowledge of these factors it is possible to control the effects on the pavement by subgrade preparation and slab design. The importance of the subgrade soil properties is especially observed in the performance of oil and gravel road surfaces, since a portion of the soil is incorporated with the surfacing material. By using the soil profile as a basis for correlating the different types of road deterioration or condition, it has been

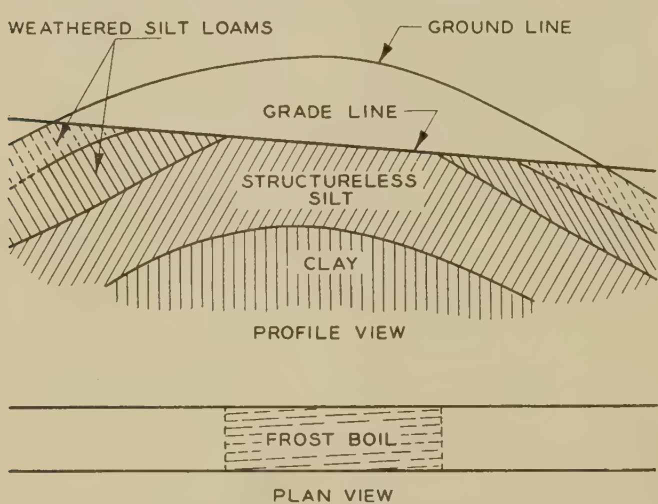

FIGURE 8.—FROST BOIL IN STRUCTURELESS SILT

possible to determine under what conditions and upon what soil layer or soil material of a soil type the different kinds of road failures can be expected.

The correlation of the condition of roads in service with the soil profile and the determination of the physical characteristics of the layers composing the profile as disclosed by laboratory tests form the basis for the grouping of subgrade soils according to performance. The success obtained by these correlation studies shows that the soil profile is a reliable, scientific, and economical means of approaching and solving subgrade soil problems. Furthermore, the soil profile offers a method for the practical application of subgrade research to highway construction.

DETERMINATION OF THE SOIL PROFILE

The soil profile is determined by examining the soil in its natural field condition. This work is best accomplished by examining excavations, road cuts, etc., but the method of using a soil auger is the most common. There is no definite rule to follow in making these examinations, except that the soil should be examined at intervals close enough to determine the soil type and by borings deep enough to penetrate the more or less non-uniform layers of soil or soil material.

According to Doctor Marbut (*4*) the examination of the soil section, after the locality has been determined upon, should proceed in a systematic way somewhat as follows:

1. Texture.[3]—The successive layers or horizons differing in texture, or in fineness or coarseness of the material, should be carefully examined. The examination should extend to a depth of at least 5 or 6 feet. The texture of each layer and its thickness should be described.

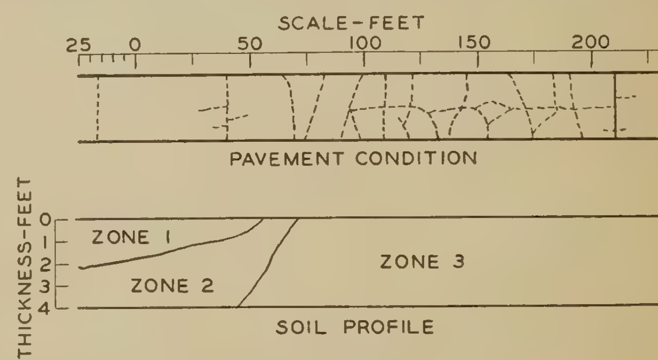

FIGURE 9.—RELATION OF VARIOUS SOIL ZONES TO PAVEMENT CONDITION

2. Color.—The successive layers which can be differentiated according to differences in color, should be noted. Each layer should be described and its thickness given.

3. Structure.—The several layers that differ according to structure should be examined carefully, structure being defined as the kind and size of soil particle aggregation. Special note should be made of horizons with (*a*) fine granular structure (granules about the size of bird shot or smaller); (*b*) coarse granular structure (granules ranging up to half an inch or more in diameter and usually more angular or irregular in shape than the granules making up the fine granular structure); (*c*) layered or platy structure, in which the material splits into thin plates (not to be confused with stratification); (*d*) buckshot structure, in which the soil on drying breaks up into angular fragments (found to characterize heavy clays usually having a considerable percentage of lime); and (*e*) single-grain structure in which the material is like flour or sand with no aggregation of particles.

4. Consistency.—A determination should be made of the successive layers or horizons differing in consistency (stickiness, friability, plasticity). A description of each should be given and its thickness noted.

5. Compactness.—The relative compactness of the several layers should be determined, as measured by the degree of resistance to the penetration of a pointed instrument.

6. Cementation.—It should be determined whether or not resistance to penetration is due in any horizon to

[3] The terms used by the pedologist in describing the various layers of the soil profile are defined in Appendix A of this report.

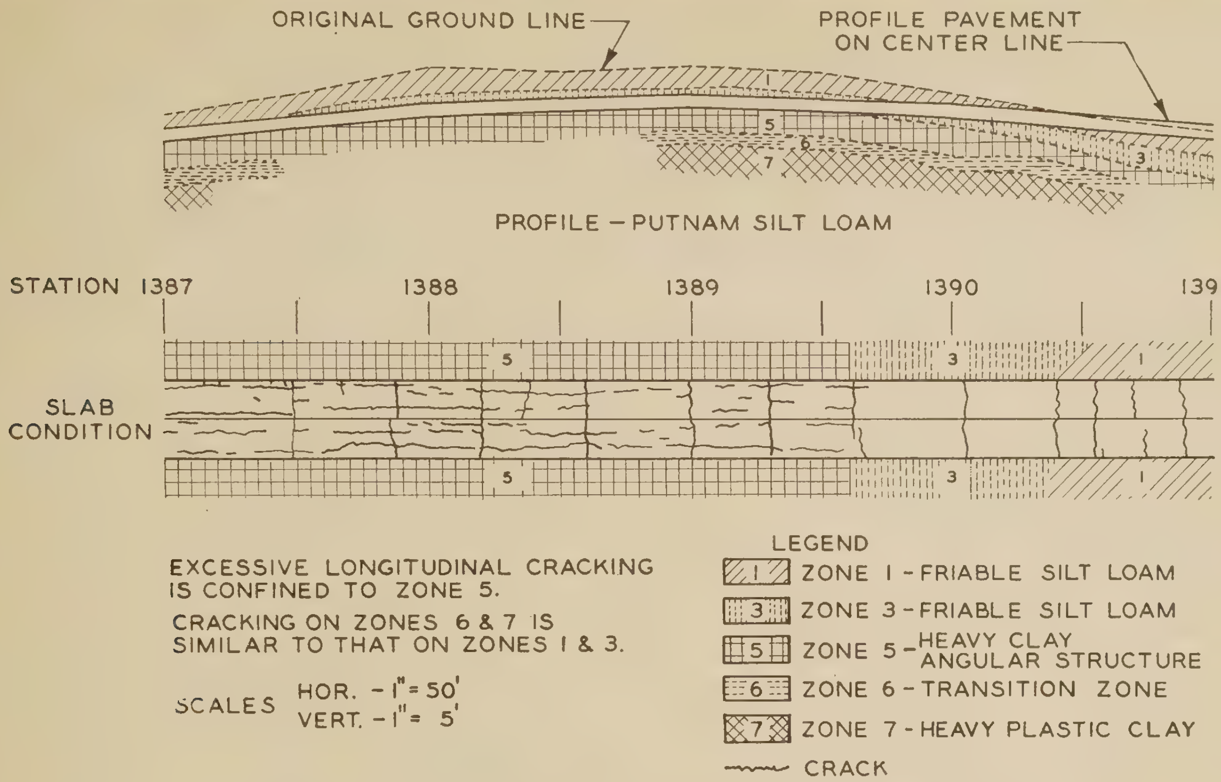

Figure 10.—Cracking Which Developed During Curing Period

cementation. If so, the probable cementing material (light colored or reddish, very strongly cemented or weakly cemented) should be ascertained.

7. *Chemical composition*.—While the determination of the chemical composition of the various parts of the soil section or profile can not be performed in the field by the usual field methods, there are certain features that may be determined in at least a qualitative way. Field examination can detect the presence of horizons with concentrations of organic matter or of salts of the alkalies and alkaline earths.

The organic matter referred to here is that contained in the soil and not that lying on the soil. Of this there are two kinds to be looked for. (*a*) The organic matter in the surface soil is recognized by the dark color, and the approximate relative percentage present is indicated by the intensity of the dark color. The determination of the thickness of the dark-colored layer in the virgin soil is very important. (*b*) In some soils, usually confined to regions with a cool, moist climate, there is present, at a depth ranging from 6 inches or less to somewhat more than a foot, a layer of brown or coffee-brown organic matter forming a film 6 or 8 inches in thickness.

The salts of the alkalies and alkaline earths accumulate in the soil under favorable conditions. Since the work here contemplated is general and the soil characteristics dealt with are those of wide regional distribution, we may practically neglect all salts except the carbonate of lime. The more soluble salts constituting what is usually known as "alkali" are present in relatively small areas and may be neglected, or the soils in which they occur may be designated merely as alkali soils.

Horizons of lime carbonate accumulation may be identified readily by anyone and should be looked for where the rainfall is less than 17 to 18 inches per year in cool to cold climates and 30 inches per year in hot or very warm climates. The unweathered material beneath the soil in any region, arid, subhumid, or humid may have a high percentage of lime carbonate, but such material should not be confused with the horizon of true lime carbonate accumulation.

Sesquioxides accumulate in the soil under favorable conditions. Since accumulations of aluminum hydroxide are not readily identified by the usual field methods these may be left out of consideration. We are concerned, therefore, with accumulations of iron oxides.

Iron oxides occur in two forms: The first form consists of accumulations of finely divided or colloidal iron oxide (hydroxide). The degree of concentration may be determined, within a rather wide range of error, by the intensity of the red color. The existence of red horizons in the soil profile should be noted and should be illustrated with samples, even though they be small. The second type consists of accumulations of iron oxide concretions or large masses, usually porous or slaglike. This statement does not refer to ironstone slabs or ferruginous sandstone layers which may be found in many places in the parent geological formations. The accumulations referred to here are to be found either in the B horizon or at the top of the C horizon. In hot countries they take the form of thick masses of porous slaglike iron oxide lying at depths ranging from somewhat less than 3 to more than 15 feet. They may consist of fragments scattered over the surface.

MAPPING THE SOIL PROFILE

The detailed mapping of soil profiles is accomplished in the following manner:

1. Vertical soil sections are examined at frequent intervals and classified into layers according to the method described above (*4*). The interval at which soil examinations are made depends on the uniformity of the soil examined.

2. The limits of the various layers are plotted as shown in Figure 11, A, alternate numbers being used to indicate the layers. At test hole No. 1, the con-

secutive layers are numbered 1, 3, 5, etc., so as to allow the inclusion of any other layer which might enter the profile within the section mapped. At test hole No. 4, layer 2 is mapped between layers 1 and 3. This requires the examination of another test hole (No. 5), or perhaps several, between Nos. 3 and 4, to determine the horizontal limits of layer 2.

3. The profile is completed by connecting the points marking the limits of the layers as shown in Figure 11, B.

DETAILS OF THE SUBGRADE SURVEY DESCRIBED

The purpose of the subgrade survey is to furnish the engineer with significant information on the following subjects:

1. The final location of the road both vertically and horizontally.
2. The selection of suitable fill material.
3. The design of the roadway section.
4. The design and location of ditches, culverts, and drains.
5. The need for subgrade treatment and the type required.
6. The selection of the type of road surface and its design.

The subgrade survey consists of three parts: (*a*) The determination of the soil profile, (*b*) the determination of the physical properties of the soils included in the profile, and (*c*) the mapping of the profile in order to supply information important to road design.

The determination of the physical properties of soils and soil materials and their grouping according to performance has been discussed in previous reports (*8*, *13*).

The method of making the subgrade survey depends on the type of information required. Two types of subgrade survey are made, (*a*) surveys to furnish information with respect to roads in service, and (*b*) surveys to furnish information with respect to the design of new roads.

EQUIPMENT

The following equipment is required to make subgrade surveys:

One 3-foot soil auger and three 3-foot extensions as illustrated in Figure 12.
Two small pipe wrenches.
One light pick.
One shovel.
A supply of sample bags.
A supply of tags for marking samples.
A ball of twine.
One engineer's level.
One hand level.
One 12-foot level rod, three-section.
One 100-foot metallic tape.
One 12 by 15 inch strip of stiff cardboard.
One roll of 20-inch cross-section paper, 10 divisions to the inch each way.
Notebooks.
A supply of survey stakes.
One camera and supply of films.
A supply of keel.

SUBGRADE SURVEY TO OBTAIN INFORMATION REGARDING ROADS IN SERVICE

The section for study having been selected, the procedure is carried out in the following manner:

1. The section is staked out, the original construction stations being used if possible. Arbitrary stations will serve the purpose when it is not convenient to locate original stations.

2. Cross-sections are taken every 50 feet along the center line or oftener if topography requires, and for a distance of 150 feet on each side of the center line. Elevations are obtained with an engineer's level to the nearest tenth of a foot. The accuracy of an engineer's level is necessary for the construction of center-line and bank-line profiles, but a hand level is sufficiently accurate for the topography adjacent to the highway. An assumed elevation may be used as a bench mark.

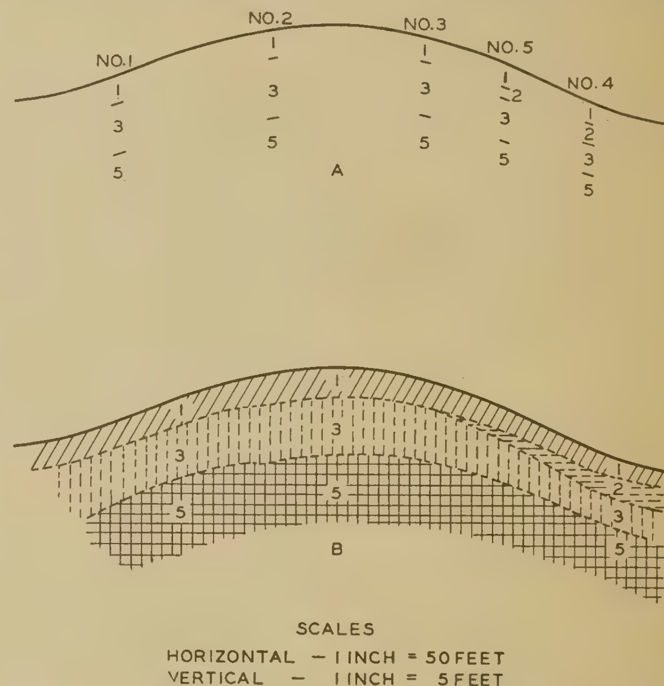

FIGURE 11.—EXAMPLE OF SOIL PROFILE MAPPING, ILLUSTRATING THE DETERMINATION OF LIMITS OF INTERMEDIATE LAYERS BY SUPPLEMENTARY BORINGS

3. A plan of the roadway is drawn, showing the type of pavement, type of failure, portion of roadway which is built over an older road, if any, and any special construction. A scale of 50 feet to the inch is used.

4. Cross-section notes are plotted to the same scale as the plan of roadway and contours are drawn in by interpolation.

5. The bank-line profiles are drawn and the center-line profile is projected upon them, as well as the grade line of the preexisting road, if any. The horizontal scale is 50 feet to the inch. The vertical scale is 5 feet to the inch. Ordinary cross-section paper, 20 inches wide with 10 divisions to the inch each way, is the most convenient type of record sheet. Sheets are cut to a length of 30 inches, folded two ways and clamped to the sheet of stiff cardboard 12 inches wide and 15 inches long.

6. The soils are mapped and the profiles plotted according to the procedure outlined previously. The desired information is obtained and recorded in the following manner:

a. The back slopes are scraped down so that the original undisturbed material is exposed and the limits of the various layers are plotted on the prepared profile sheet. This work is supplemented by soil-auger boring so that a profile is obtained to a depth of at least 3 to 5 feet below the center-line grade. Any variations in moisture content are specially noted. The depth will vary with the uniformity of the soil layers or soil material. The elevations of the limits of the different layers in the exposed back slopes are obtained by means of a hand level, the elevation of the center line being used as a bench mark.

b. In a separate notebook each layer is described in detail according to the nomenclature of the committee on terminology (*3*). (See Appendix A.) The relative imperviousness or porosity is also recorded.

c. Examinations of the soil are made every 50 feet or less, depending upon the uniformity of the profile. (See discussion of Figure 11.)

d. On the plan of the roadway are plotted the limits of the various soil layers found directly under the surfacing material. When the roadway is cut through uniform layers of soil material, the limits are obtained by constructing cross-sections from the bank-line profiles. When the roadway is constructed of fill material or cut through a heterogeneous soil material, the limits are determined by soil-auger borings in the shoulder.

7. A 5-pound sample of soil from each layer is obtained from the exposed back slopes with a pick and shovel, placed in a canvas bag, tied securely, marked with proper identification, and shipped to the laboratory. A sufficient number of samples are taken to determine the range in test results for what appears to be the same layer.

8. Photographs are taken illustrating the condition of the pavement, the shoulders, the back slopes, the ditches, and the appearance of the soil layers.

9. The data collected in the manner described above are analyzed, together with the laboratory test results, and information on the following subjects is developed:

a. The relation that exists between the pavement condition, the field characteristics of the soil, and the physical properties of the soil as determined in the laboratory.

b. The possible reasons for failure.

c. Possible curative measures for the case under examination.

d. Preventive measures which may be applied in the future.

A sample report of a subgrade survey of a road in service is given in Appendix B of this report to illustrate the above procedure.

SUBGRADE SURVEY TO OBTAIN INFORMATION WITH RESPECT TO THE DESIGN OF A NEW ROAD

Before starting a subgrade survey of this kind the engineer should make a study of all the existing information on the soil types in that vicinity. Wherever maps prepared by the Bureau of Soils[4] are available, they should be carefully studied, and the limits of the various soil types and their characteristics should be noted. It must be kept in mind that the detail to which such maps are carried is not particularly adapted to a subgrade survey. Nevertheless they give a clear idea of the variations which will be encountered. Where this information is not available, a reconnaissance survey should be made of the soil materials on existing highways which parallel the new highway, noting the changes in soil as shown in exposed cuts. The notes should include a description of each soil type according to the nomenclature of the committee on terminology. (*3*) The value of the information obtained from this rough survey lies in the fact that similar soil conditions may be expected to accompany similar topographic features.

After this information has been digested the survey proceeds in the following manner:

1. The profile of the ground line and the proposed grade line are constructed on the same type of sheet as was specified for subgrade surveys of existing pavements.

2. Borings are made with a soil auger at frequent intervals and each soil type is classified into layers, as described under the heading "Determination of the Soil Profile" (p. 54).

a. *Spacing of borings*.—The spacing of the borings will vary with the uniformity of the profile and the topography. A convenient interval, such as the even stations, may be assumed at the beginning. This interval may be varied under the following conditions: (1) If the profile is uniform, the interval may be increased. (2) When the character of the profile changes, intermediate borings should be made until it is clear that all variations have been mapped. (3) Where topography is rolling and grade changes rapidly from cut to fill, borings are necessary only in the cuts. (4) Where the original ground line or old road grade is to be covered with fill material, no examination is necessary except to determine the character of the support. If the fill material is to be obtained from borrow ditches along the road, the soil should be examined to the entire depth of the borrow.

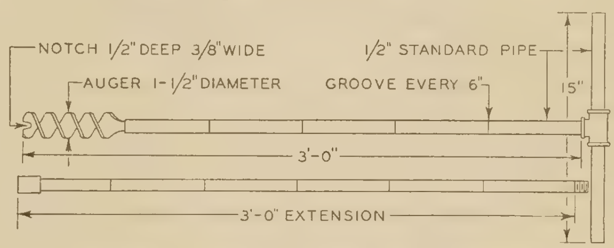

FIGURE 12.—SOIL AUGER AND EXTENSION

b. *Depth of borings*.—The borings should generally be carried to a depth of at least 3 feet below the grade line. The depth may vary in accordance with the following stipulations: (1) When the road lies within the uniform layers of the soil profile, the boring should extend down to the first layer below the ditch line which would block percolation, or through a pervious layer which would carry water. (2) When fill material is to be borrowed from ditches alongside the road, the boring should extend at least to the estimated depth of borrow. (3) In the study of frost action the borings should extend to the mean depth of frost in those soil materials showing a high affinity for frost accumulation (see p. 53) and in localities where high water tables prevail.

c. When the located line is over an old road, the soils are mapped by examining the exposed cuts. This work is supplemented by borings.

3. A notation is made of the direction of surface drainage with respect to the proposed roadway.

4. The data obtained from the borings is plotted on the prepared profile sheet. On this sheet are indicated the limits of the several types and layers, the relative moisture content at various depths, and the location of culverts and drains.

5. In a separate notebook the field characteristics of each layer are described according to the nomenclature prepared by the committee on terminology. The relative imperviousness or porosity of each layer is indicated.

6. From each layer and type encountered at least a 2-pound sample of soil is taken for laboratory classification. A sufficient number of samples should be obtained to determine the range in test results for what

[4] The former Bureau of Soils is now a part of the Bureau of Chemistry and Soils.

appears to be the same layer. These samples are placed in canvas bags, tied securely, marked with station number and layer, and shipped to the laboratory. The boring of more than one hole may be necessary to obtain the required sample.

7. Recommendations regarding the design of the road surface are made on the basis of the known behavior of pavements for which the conditions of soil, climate and topography are similar.

8. After the road is graded, a final check is made on the soil as exposed by grading operations.

The final form of a subgrade survey sheet submitted to the engineer in charge of design is included in Appendix B of this report.

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APPENDIX A

TERMS IDENTIFYING SOILS IN THE PROFILE

The following terms [1] are among those used in describing the various layers of the soil profile. They definitely identify the field characteristics of the soil material as found in its natural state. In the compilation of these terms the report of the committee on terminology of the American Soil Survey Association was largely used.

[1] The terms describing structure, consistency, compactness, cementation, and chemical composition have not yet been standardized, nor have they been adopted by the division of soil survey, Bureau of Chemistry and Soils. They may be considered tentative and approximate. More accurate definitions are not yet available.

TEXTURE

Texture is a term indicating the size of the individual soil grains or particles and the proportions of material of each size present in any given case.

As the soil is usually made up of particles of widely varying size, the textural terms express the average effect or the combined effect of all these grain sizes. They may indicate the predominance (in quantity or in textural effect) of a certain group of grains.

Texture is determined by mechanical analysis, a laboratory process of separating the soil into groups of grain sizes. The system of mechanical analysis used by the Bureau of Chemistry and Soils separates the soil material into seven grain sizes or "separates" having the following sizes and names:

2 to 1 millimeter, fine gravel.
1 to 0.5 millimeter, coarse sand.
0.5 to 0.25 millimeter, sand.
0.25 to 0.10 millimeter, fine sand.
0.10 to 0.05 millimeter, very fine sand.
0.05 to 0.005 millimeter, silt.
Below 0.005 millimeter, clay.

In the following paragraphs are given the proportions of certain of the grain sizes found in the major soil textures:

Sands contain less than 20 per cent of silt and clay. (Include coarse, fine, and very fine sands.)

Sandy loams contain from 20 per cent to 50 per cent of silt and clay but do not have over 15 per cent of clay. (Include coarse, fine, and very fine sandy loams.)

Loams have more than 50 per cent of silt and clay combined but have less than 50 per cent of silt and less than 20 per cent of clay.

Silt loams have more than 50 per cent of silt and less than 20 per cent of clay.

Clay loams have more than 50 per cent of silt and clay combined but less than 50 per cent of silt and between 20 per cent and 30 per cent of clay. (Include sandy clay loams, clay loams, and silty clay loams.)

Clays have more than 50 per cent of silt and clay combined and more than 30 per cent of clay. (Include sandy clays and silty clays.)

In the field texture is determined by the feel of the soil mass when rubbed between the fingers. The following statements give the obvious physical characteristics of the basic textural groups:

Sand.—Sand is loose and granular. The individual grains can readily be seen or felt. Squeezed in the hand when dry it will fall apart when the pressure is released. Squeezed when moist, it will form a cast, but will crumble when touched.

Sandy loam.—A sandy loam is a soil containing much sand but having enough silt and clay to make it somewhat coherent. The individual sand grains can readily be seen and felt. Squeezed when dry, it will form a cast which will readily fall apart, but if squeezed when moist a cast can be formed that will bear careful handling without breaking.

Sands and sandy loams are classed as coarse, medium, fine, or very fine, depending on the proportion of the different sized particles that are present.

Loam.—A loam is a soil having a relatively even mixture of the different grades of sand and of silt and clay. It is mellow with a somewhat gritty feel, yet fairly smooth and slightly plastic. Squeezed when dry, it will form a cast that will bear careful handling, while the cast formed by squeezing the moist soil can be handled freely without breaking.

Silt loam.—A silt loam is a soil having a moderate amount of the fine grades of sand and only a small amount of clay, over half of the particles being of the size called "silt." When dry it may appear quite cloddy, but the lumps can be readily broken, and when pulverized it feels soft and floury. When wet the soil readily runs together and puddles. Either dry or moist it will form casts that can be freely handled without breaking. If squeezed between thumb and finger it will not "ribbon" but will give a broken appearance.

Clay loam.—A clay loam is a fine-textured soil which breaks into clods or lumps that are hard when dry. When the moist soil is pinched between the thumb and finger it will form a thin ribbon which will break readily, barely sustaining its own weight. The moist soil is plastic and will form a cast that will bear much handling. When kneaded in the hand it does not crumble readily but tends to work into a heavy compact mass.

Clay.—A clay is a fine-textured soil that forms very hard lumps or clods when dry. When the moist soil is pinched out between the thumb and fingers it will form a long, flexible ribbon.

Gravelly or stony soils.—All of the above classes of soils, if mixed with a considerable amount of gravel or stone, may be classed as gravelly sandy loams, gravelly clays, etc., as stony

sandy loams, stony loams, etc., or as sandy clay loams, sandy clays, etc.

Floury.—Fine-textured soil consisting predominantly of silt (or flocculated clay with aggregates of silt size) which when dry is incoherent, smooth, and dust-like.

Gritty.—Containing a sufficient amount of angular grains of coarse sand or fine gravel, so that these dominate the "feel." Usually applied to medium-textured soils (loams) where the actual quantity of these coarse grains is rather small.

Heavy (textured).—Applied to soils of fine texture in which clay predominates, with dense structure and firm to compact consistency. The term is also applied to soils containing a somewhat higher proportion of the finer separates than is typical of that textural class (as a "heavy sandy loam").

Light (textured).—Applied to soils of coarse to medium texture with very low silt and clay content, incoherent, single-grained structure, and loose consistency. The term is also applied to soils containing somewhat higher proportions of the coarser separates than is typical of that textural class (as a "light loam").

Sharp.—Containing angular particles in sufficient amount to dominate the "feel." Abrasive.

Smooth.—Containing well-rounded coarser particles and a predominance of the finer separates. Not abrasive.

COLOR

In order to recognize the same soil in some other locality, the color should be clearly stated. This statement should give the range of color included, for classification purposes. By range of color is meant such terms as black or dark brown, brown to reddish brown, etc. The color may indicate the drainage conditions under which the soil was formed and the chemical composition of the soil.

In using such terms as grayish brown, brownish gray, etc., the adjective is recognized as a modifying term. The grayish brown soil is a brown soil with a grayish cast sufficiently noticeable to require recognition; the brownish gray soil is a gray soil with a brown cast.

Other color terms are as follows:

Mottled.—The presence of spots, streaks, or splotches of one or more colors in a soil mass of another predominant color. In mottled soils the colors are not mixed and blended, but each is more or less distinct in the general ground color. In color descriptions the ground color and the color of the included spots should be designated. Mottling is usually but not necessarily associated with poor drainage. The use of the term should not be confined to poorly drained soils but should be applied wherever the term fits.

Marbled.—The presence of two or more distinct colors in approximately equal amounts not blended but more or less mixed in occurrence in the soil mass. In a marbled soil there is no general or predominant color, as in the case of a mottled soil.

Spotted, speckled, streaked, variegated.—Such terms can be used when their generally accepted meaning describes the color distributions that occur in the soils.

STRUCTURE

The term "soil structure" expresses the arrangement of the individual grains and aggregates that make up the soil mass. The structure may refer to the natural arrangement of the soil when in place and undisturbed (as structural profile) or to the soil at any degree of disturbance. The terms used indicate the character of the arrangement, the general shape and the size of the aggregates, and in some cases may indicate the consistency of those aggregates.

Adobe structure.—This term describes a soil which on drying cracks and breaks into irregular but roughly cubical blocks. The cracks are usually wide and deep and the blocks are from 20 to 50 or more centimeters across. (Adobe soils are usually heavy-textured and high in content of colloidal clay.)

Amorphous structure.—A soil of fine texture having a massive or uniform arrangement of particles throughout the horizon. Structureless. Found only in soils of finest texture, where individual grains can not be recognized.

Clod (or cloddy) structure.—Aggregates of irregular, angular shape, usually 4 centimeters or more in diameter and of a hard consistency.

Fine cloddy structure.—When most of the clods are close to the minimum size.

Coarse cloddy structure.—When most of the clods are 10 centimeters or more in diameter.

Columnar structure.—A natural arrangement of the soil mass in more or less regular columns separated by vertical cleavage lines, and usually broken by horizontal cracks into sections with longer vertical than horizontal axes, the tops of the columns being rounded.

Prismatic columnar structure.—Term used when the sections are very regular in size, straight-sided, with the vertical axes much longer than the horizontal axes and the tops of the columns flat.

Crumb structure.—Porous aggregates of irregular shape, rarely over 2 centimeters in diameter and of a medium to soft consistency.

Fine crumb structure.—Crumbs 5 millimeters or less in diameter.

Coarse crumb structure.—Crumbs 2 centimeters or more in diameter.

Crust (or crusted) structure.—This term is used where the upper or surface horizon coheres into plate or crust distinct from the horizon immediately below it.

Crust-mulch structure.—An arrangement where a surface crust is underlain by a horizon of loose, incoherent particles of mealy, crumb, or granular structure.

Fluffy structure.—A surface condition where the aggregates are loose, of light weight and fine texture, with no cohesion or evidence of arrangement; floury.

Dense structure.—Having a minimum of pore space and an absence of any large pores or cracks. Approaching amorphous.

Granular structure.—Aggregates varying in size to 2 centimeters in diameter, of medium consistency, and more or less subangular or rounded in shape.

Fine granular structure.—Aggregates under 5 millimeters diameter.

Coarse granular structure.—Aggregates close to maximum size.

Honeycomb structure.—A natural arrangement of the soil mass in more or less regular five or six sided sections separated by narrow or hairline cracks. Usually found as a surface structure or arrangement.

Hardpan.—An horizon of accumulation that has been thoroughly cemented to an indurated, rock-like layer that will not soften when wet. The term hardpan is not properly applied to hard clay layers that are not cemented, nor to those layers that may seem indurated when dry but which soften and lose their rock-like character when soaked in water. The true hardpan is cemented by materials that are not readily soluble, and is a hard layer that definitely and permanently (in nature) limits downward movement of roots and water.

Clay pan.—An horizon of accumulation or a stratum of stiff, compact, and relatively impervious clay. The clay pan is not cemented, and if immersed in water can be worked to a soft mass. Its presence may interfere with water movement or root development the same as a true hardpan. It is more difficult to overcome, for, whereas a hardpan can be shattered by explosives, the clay pan, after breaking by any means, will run together and re-form as soon as thoroughly wetted. The distinction between hardpan and clay pan is an important one in the soil classification.

Laminated structure.—An arrangement of the soil mass in very thin plates or layers, less than 1 millimeter in thickness, lying horizontal or parallel to the soil surface. Usually medium to soft consistency.

Massive structure.—A soil mass showing no evidence of any distinct arrangement of the soil particles. Structureless. May be found in soils of any structure.

Mealy structure.—A crumb-like structure in which the aggregates are of soft to very soft consistency and usually less than 5 millimeters in diameter.

Nut structure.—Compact aggregates, more or less rounded in shape, of hard to medium consistency, and from one-half to 4 centimeters in diameter.

Fine nut structure.—Aggregates below 1 centimeter in diameter.

Coarse nut structure.—Aggregates over 3 centimeters in diameter.

Plate (or platy) structure.—An arrangement of the soil mass in plates or layers 1 to 5 millimeters or more in thickness, lying horizontal or parallel to the soil surface. Usually medium to hard consistency.

Single-grained structure.—An incoherent condition of the soil mass with no arrangement of the individual particles into aggregates. Structureless. Usually found in soils of coarse texture.

Structureless.—Without any discernible structure or arrangement of the soil particles into aggregates. This condition is better expressed by the terms single-grained, massive, amorphous, etc.

Vesicular structure.—A soil horizon or soil aggregate containing many small rounded cavities smooth on the inside as though formed by gas bubbles.

CONSISTENCY

"Soil consistency" is a term expressing the degree of cohesion of the soil particles and the resistance offered to forces tending to deform or rupture the aggregates. Consistency and structure

are closely related and frequently interdependent. The terms expressing consistency and structure are distinct, however, and need not be confused or used with double meaning. A study of published reports shows a general use of terms expressing both the consistency and the structure in nearly all soil descriptions.

Brittle.—A soil which when dry will break with a sharp, clean fracture. If struck a sharp blow, it will shatter into cleanly broken hard fragments.

Mellow.—Soil particles or aggregates are weakly adhered in a rather porous mass, readily yielding to forces causing rupture. A consistency softer than friable. Without tendency to pack.

Plastic.—Readily deformed without rupture. Pliable but cohesive. Can be readily molded. Puttylike. This term applies to those soils in which at certain stages of moisture the grains will readily slip over each other without the mass cracking or breaking apart.

Soft.—Yielding readily to any force causing rupture or deformation. Aggregates readily crushed between fingers.

Sticky.—Applied to soils showing a decided tendency when wet to adhere to other materials and foreign objects.

Firm.—Resistant to forces tending to produce rupture or deformation. Moderately hard. Aggregates can be broken between fingers.

Friable.—Aggregates readily ruptured and crushed with application of moderate force. Easily pulverized or reduced to crumb or granular structure.

Hard.—Resistant to forces tending to cause rupture or deformation. Difficult or impossible to crush aggregates with fingers only.

Tenacious.—Soils showing a decided resistance to rupture. Soil mass adheres firmly.

The terms "sticky" and "tenacious" are often used as synonyms, but in soil usage the former is taken to refer to adhesion, the latter to cohesion. Both terms may be applicable to a soil at the same time.

Stiff.—Resistant to rupture or deformation. A soil stratum or horizon that is firm and tenacious, and tending toward imperviousness. Usually applied to condition of the soil in place and when moderately wet.

Tight.—A stratum or horizon that is compact, impervious and tenacious, and usually plastic.

Tough.—Resistant to rupture. Tenacious. A stratum or horizon that can be readily bored into with the auger but which requires much force in breaking loose and pulling out the core of soil.

COMPACTNESS

Compactness is the degree of resistance offered by a soil to the penetration of a pointed instrument.

Impervious.—Highly resistant to penetration by water and usually resistant to penetration by air and plant roots. Impenetrable. In field practice the term is applied to strata or horizons that are very slowly penetrated by water and that retard or restrict root penetration.

Indurated.—(See under cementation).

Loose.—Soil particles or small aggregates are independent of each other or cohere very weakly with a maximum of pore space and a minimum resistance to forces tending to cause rupture.

Cheesy.—Having a more or less elastic character, deforming considerably without rupture, yet broken without difficulty or the application of much force. (Characteristic of certain highly colloidal soils when thoroughly wet.)

Compact.—The soil packed together in a dense, firm mass, but without any cementation. Noticeably resistant to forces tending to cause rupture or deformation. Coherent. Hard.

Relative degree of compactness may be expressed by terms as slightly compact, very compact, etc.

CEMENTATION

Cementation.—A condition occurring when the soil grains or aggregates are caused to adhere firmly and are bound together by some material that acts as a cementing agent (as colloidal clay, iron or aluminum hydrates, lime carbonate, etc.).

The degree of cementation or the persistence of the cementation when the soil is wetted should be stated. Some terms indicate the permanence, as "indurated," "hardpan," etc.

Firmly cemented.—Cementing material of considerable strength requiring considerable force to rupture the mass. Usually breaks, with clean though irregular fractures, into hard fragments.

Indurated.—Cemented into a very hard mass which will not soften or lose its firmness when wet, and which requires much force to cause breakage. Rock-like.

Weakly cemented.—Term applied when cementing material is not strong, and the aggregates can be readily broken into fragments with a more or less clean fracture.

Softly cemented.—Term applied when cementing material is not strong nor evenly diffused throughout the mass. Aggregates are readily crushed, but do not break with a clean fracture.

CHEMICAL COMPOSITION

Peat soil.—Composed predominantly of organic material, highly fibrous, with easily recognized plant remains.

Muck soil.—Composed of thoroughly decomposed black organic material, with a considerable amount of mineral soil material, finely divided and with a few fibrous remains.

Leaf mold.—The accumulation on the soil surface of more or less decomposed organic remains, usually the leaves of trees and remains of herbaceous plants. The A horizon.

Alkaline soil.—A soil containing an excessive amount of the alkaline (in true chemical sense) salts.

Saline soil.—A soil containing excessive amounts of the neutral or non-alkaline salts.

Calcareous soil.—A soil containing sufficient calcium carbonate to effervesce when tested with weak (N/10) hydrochloric acid. Depending on the amounts present, these soils may be designated as slightly calcareous, strongly calcareous, etc.

Acid soil.—A soil which is deficient in available bases, particularly calcium, and which gives an acid reaction when tested by standard methods. Field tests are made by the use of litmus, of Soiltex, and of other indicators. There is no full agreement on the most satisfactory test for acidity or as to the actual character of an acid soil. The intensity or degree of acidity may be expressed by qualifying words, "strongly," "moderately," etc.

APPENDIX B

SAMPLE REPORT OF A SUBGRADE SURVEY OF A ROAD IN SERVICE

ROAD DESCRIPTION

Designation.—United States Highway No. 51, Federal aid project 82–A, situated two miles south of Jackson, Hinds County Miss.

Date constructed.—1927.

Width.—Roadway, 26 feet; pavement, 18 feet.

Pavement thickness.—Seven inches at the edges and 6 inches at the center, stations 97+00 to 108+88; 9 inches at the edges and 6 inches at the center, stations 108+88 to 112+50.

Joints.—One-half inch transverse expansion joints spaced about 30 feet apart, stations 97+00 to 108+88; tongue-and-groove longitudinal center joint with metal center strip and ½-inch transverse expansion joints spaced about 50 feet apart, stations 108+88 to 112+50.

Reinforcement.—Welded fabric placed 2 inches from the top of the slab.

Concrete design.—One part Portland cement, 2 parts local sand, and 3 parts local gravel.

Curing.—Two per cent calcium chloride admixture.

Compressive strength of concrete.—Average of sixteen 6 by 12 inch cylinders tested at the end of 28 days equaled 3,230 pounds per square inch.

Drainage.—Eight-inch bell-and-spigot concrete drain placed in west ditch at a depth of 3 feet below subgrade elevation. Trench is back filled with washed gravel as shown in Figure 1.

Reasons for survey.—(*a*) To determine if the detrimental distortion suffered by an appreciable number of the pavement slabs comprising this project was related to the subgrade, and (*b*) to determine what precautions might be taken to prevent this type of failure from occurring in pavements laid on similar subgrades in the future.

Cooperating agencies.—The Mississippi State Highway Department and the Portland Cement Association.

DATA FURNISHED BY THE SURVEY

The soil profile.—The profile consists of five layers designated as layer 1, layer 3, layer 5, layer 6, and layer 7, shown in Figure 1.

Layer 1, averaging about 18 inches in thickness, consists of a light brown to reddish or grayish brown mellow silt loam of fine crumb structure containing both black and brown iron concretions. The grayish brown color becomes rather pronounced in the lower 10 inches of layer 1 where it grades into layer 3.

Layer 1 soil is friable when dry, has a pasty consistency when wet, and according to Table 2, it has characteristics similar to those of the plastic varieties of the group A–4 subgrades.

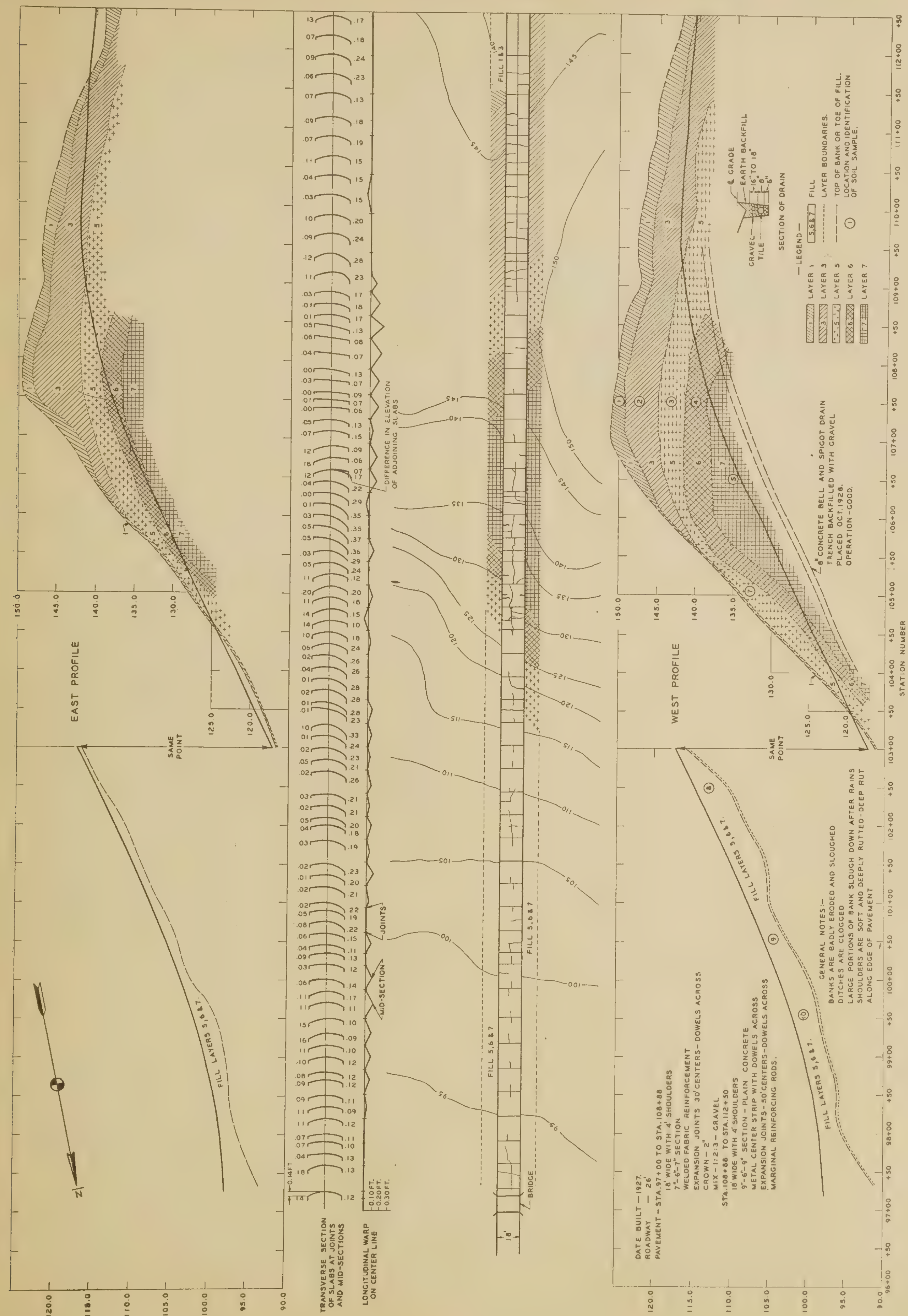


FIGURE 1.—SUBGRADE SURVEY SHEET FOR ROAD IN SERVICE

Layer 3 grades from layer 1 into a brown and gray mottled silty clay loam or silty clay of moderately compact structure. The upper 10 inches of layer 3 is a transition from layer 1, but is much more dense and compact. This compactness increases with increasing depth until at about 5½ feet layer 3 has a higher moisture content and grades into the angular-structured layer 5.

The soil of layer 3 is friable when dry and very plastic when wet and is permeable regardless of its compactness. Bank erosion, shown in Figure 2, causes the soil of layer 3 to be fissured and to assume the form of pinnacles upon drying. According to Table 2, layer 3 consists of a more plastic variety of the Group A-4 subgrade than layer 1.

Layer 3, like layer 1, contains dark-brown and black concretions in large amounts.

Figure 2.—Erosion of Back Slopes is Very Irregular and There is Considerable Sloughing of Material into Ditches

Table 1.—*Mechanical analyses of soils found in soil profile*

Identification No.	Layer	Percentage of particles having diameters smaller than—					
		2 mm.	0.5 mm.	0.25 mm.	0.05 mm.	0.005 mm.	0.001 mm.
1	1	100	98	97	86	23	12
2	3	100	97	95	93	27	15
3	5	98	92	89	89	36	16
4	6	100	100	99	81	43	17
5	7	100	100	97	93	72	47
7	5	100	100	99	98	74	(1)
8	[2] 5, 6, 7	100	100	99	92	(1)	----------
9	[2] 5, 6, 7	100	98	97	91	(1)	----------
10	[2] 5, 6, 7	100	98	97	90	(1)	----------

[1] Flocculated. [2] Fill.

Table 2.—*Physical characteristics of particles passing the 0.5 millimeter sieve*

Identification No.	Layer	Liquid limit	Plastic index	Shrinkage		Moisture equivalent		Group
				Limit	Ratio	Centrifuge	Field	
		Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*	
1	1	39	17	20	1.8	31	31	A-4
2	3	45	22	22	1.7	37	35	A-4
3	5	50	33	12	2.0	[1] 50	34	A-6
4	6	85	51	12	2.0	[1] 98	52	A-7
5	7	112	77	11	2.0	[1] 125	71	A-7
7	5	98	64	12	2.0	[1] 98	59	A-7
8	[2] 5, 6, 7	102	67	12	1.9	[1] 101	70	A-7
9	[2] 5, 6, 7	104	72	13	2.0	[1] 92	60	A-7
10	[2] 5, 6, 7	76	44	12	1.9	[1] 95	59	A-7

[1] Water-logged. [2] Fill.

Layer 5 consists of brown and gray mottled heavy, plastic, sticky clay, with well-defined particles of angular structure about one-eighth inch in diameter or larger, which have a wet, shiny, and slick surface. This layer is very much wetter than overlying zones, is open-structured, and permits the movement of water. In some portions of layer 5 bright red mottlings are found. According to Table 2, the soil of this layer has characteristics common to the highly plastic varieties of the Group A-6 subgrades (sample 3), but when calcium compounds are present in appreciable amounts (sample 7), it exhibits characteristics similar to those of the Group A-7 subgrades.

Layer 6 grades from layer 5, at a depth of about 9 feet below the ground surface, into a very sticky plastic clay, bluish gray in color, mottled with brown. As shown in Figure 3 it exists in the face of cuts as slick and shiny angular particles or clods, ¼ inch to 2 inches in the longest dimension. These clods, the larger of which are dense and have a fibrous structure, are similar to putty in consistency and are easily crushed. Water moves through this layer at a rate much slower than it moves through layer 5. The characteristics of the soil of this layer are similar to those of the Group A-7 subgrades.

Layer 7, at a depth of about 10 or 11 feet below the ground surface, grades gradually from layer 6 into a bluish gray, sticky, plastic and very dense indurated clay containing both black and brown stains. In cut faces this material, similar to that of layer 6, exists as clods or chunks, fibrous in structure and easily crushed. The soil of layer 7 is impervious, holds absorbed water tenaciously, shrinks in appreciable amount when dried, slakes readily in the presence of water and exhibits characteristics generally indicative of the Group A-7 subgrades.

Figure 3.—Right Bank at Station 105+16 Showing Structure of Layer 6 Material, and Sloughed Material on Surface

The soils of layers 5, 6, and 7 effervesce strongly when treated with dilute hydrochloric acid, thus indicating the presence of lime in appreciable amount.

Condition of shoulders and banks.—The shoulders are very soft and badly rutted as shown in Figure 4, at those locations where the road traverses the clays of layers 5, 6, and 7. In the same locations the back slopes are badly eroded (see Figure 2) and after rains are apt to slide and clog the side ditches.

Condition of pavement.—Typical pavement condition with respect to both slab distortion and cracking is disclosed by Figure 5 and Tables 3, 4, and 5.

The longitudinal warp referred to in Table 3 is defined as the difference between the average elevation of the ends of the slab and the elevation at a point midway between the ends of the slab measured at the center line. Corrections were made for vertical curvature.

The transverse warp referred to in Table 4 is defined as the present crown of the pavement at the transverse joints subtracted from the original which, exclusive of superelevated curves, was assumed to be 2 inches.

Warp, either transverse or longitudinal less than 0.03 ft. is neither shown on Figure 1 nor included in the computation of the averages in Tables 3 and 4. The apparent bumps appearing in the pavement at the transverse joints and caused by the distortion of the slabs longitudinally is illustrated in Figure 4. An analysis of the longitudinal and transverse cracking is given in Table 5.

Table 3.—*Longitudinal warp on center line*

Layer	Length	Total warp	Maximum warp	Total slabs	Number of slabs warped	Percentage of slabs warped	Average [1] warp per slab	Average [2] warp per slab
	Feet	*Feet*	*Feet*				*Feet*	*Feet*
5, 6, 7	497	1.50	0.18	16	13	81	0.12	0.09
5, 6, 7, fill	664	.97	.10	21	16	76	.06	.05
3 [3]	311	.04	.04	6	1	17	.04	.01

[1] Based on number of slabs warped.
[2] Based on total number of slabs.
[3] Center-joint section.

Table 4.—*Transverse warp from center line*

Layer	Number of joints	Total warp	Maximum warp	Average warp
		Feet	*Feet*	*Feet*
5, 6, 7	16	3.03	0.23	0.19
5, 6, 7, fill	23	2.78	.15	.12
3 [1]	5	.64	.13	.13

[1] Center-joint section.

Table 5.—*Longitudinal and transverse cracking*

Layer	Length	Number of joints	Transverse				Londitudinal		
			Number	Length	Length per slab	Average slab length	Length	Per cent	Length per slab
	Feet			*Feet*	*Feet*	*Feet*	*Feet*		*Feet*
5, 6, 7	497	16	8	147	9	21	225	45	14
5, 6, 7, fill	650	21	5	89	4	25	269	41	13
3 [1]	311	6	9	162	27	21			
5 [1]	51	1	3	58	58	13			

[1] Center-joint section.

Figure 4.—Shoulders are Rutted to a Depth of 4 Inches Below Bottom of Slab

SUMMARY

The results obtained from the survey may be summarized as follows:

1. Distortion both longitudinal and transverse occurred in greatest amount in the pavement slabs 18 feet wide and about 30 feet long laid on soil layers 5, 6, and 7 when in cut, and in but slightly less amount in similar slabs laid on mixtures of layers 5, 6, and 7 used in fills.

2. The slabs containing a center joint and transverse joints spaced about 50 feet apart laid primarily on layers 1 and 3 suffered warping in very much less amount than the slabs about 30 feet long laid on layers 5, 6, and 7.

3. The slabs of another pavement (18 feet wide and about 30 feet long) separated from layers 5, 6, and 7 of these soils by several feet of good top soil did not suffer distortion in detrimental amount.

4. Longitudinal cracks occurred only in those slabs constructed without center joint, and were either short cracks beginning at expansion joints or cracks extending throughout the lengths of the slabs. In every case, however, the longitudinal cracks were situated above dowels extending across the transverse joints. In many instances a longitudinal crack was found over one dowel on one side of a joint and over the next dowel on the other side of the joint.

Figure 5.—Warped Slabs. Transverse Joints are Marked With Sticks

5. More transverse cracks occured in the slabs about 50 feet long than in the slabs 30 feet long. The greatest concentration of combined longitudinal and transverse cracking occured in the slabs without center joint laid in cut on a subgrade consisting of either layer 7 or a combination of layers 5, 6, and 7. (See station 104+50 to station 106+50, Figure 1.)

6. Longitudinal cracking was not important in the slabs which warped in greatest amount. Even at several joints where the slabs had faulted in amounts equal to as much as 2 inches no cracks occurred above the dowels.

7. Neither the longitudinal nor the transverse cracks were of appreciable width. This fact indicates that the mesh reinforcement has served to prevent the separation of the cracked fragments of slabs.

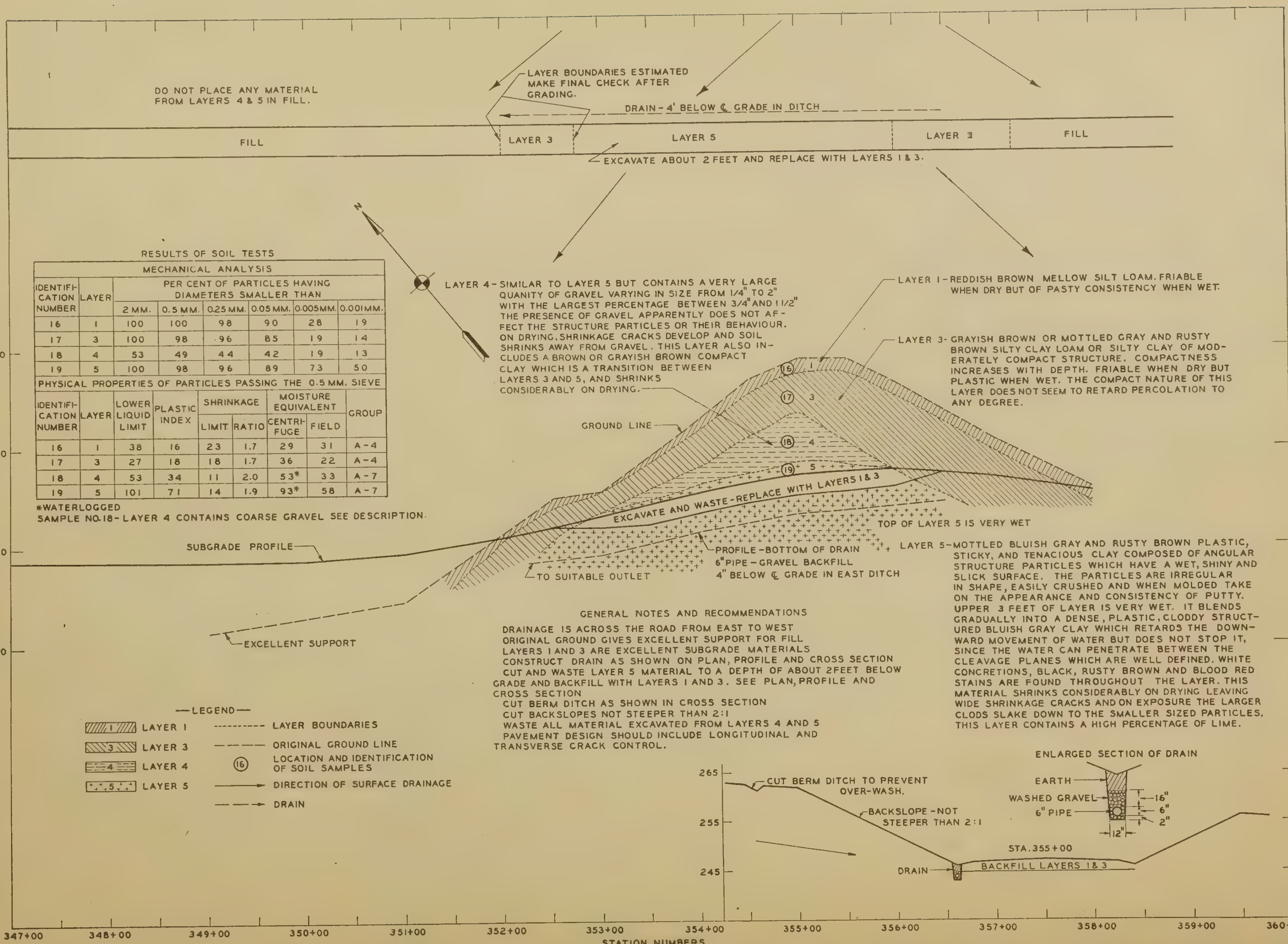
8. Generally the subgrade soil was found to be in a very soft and wet condition. Frequently when the filler was removed from the expansion joints water rose quickly to the surface of the pavement.

9. The drain serves to carry a large flow of water for several days after rains, but does not eliminate the slab movements or prevent the subgrade from becoming wet.

10. The sliding of layers, 5, 6 and 7 when occurring in the face of cuts constitutes a serious maintenance problem. In this connection a scrutiny of records covering a period of 50 years, which were furnished by a railway company operating in this region, indicates that when used in fills the soils of these layers are likely to prove troublesome until the slope becomes approximately 1:5.

SUBGRADE SURVEY SHEET GIVING INFORMATION FOR THE DESIGN OF NEW ROADS

Figure 6 is an example of the final form of survey sheet submitted to the engineer in charge of design. This form was prepared from a survey of approximately 14 miles of graded highway in Mississippi. The recommendations are based on data furnished by the behavior of existing pavements on similar subgrades. The location, grades and type of surfacing had already been established on this highway.



RESULTS OF SOIL TESTS

Identification number	Layer	Mechanical analysis: Per cent of particles having diameters smaller than 2 mm.	0.5 mm.	0.25 mm.	0.05 mm.	0.005 mm.	0.001 mm.
16	1	100	100	98	90	28	19
17	3	100	98	96	85	19	14
18	4	53	49	44	42	19	13
19	5	100	98	96	89	73	50

PHYSICAL PROPERTIES OF PARTICLES PASSING THE 0.5 MM. SIEVE

Identification number	Layer	Lower liquid limit	Plastic index	Shrinkage: Limit	Shrinkage: Ratio	Moisture equivalent: Centrifuge	Moisture equivalent: Field	Group
16	1	38	16	23	1.7	29	31	A-4
17	3	27	18	18	1.7	36	22	A-4
18	4	53	34	11	2.0	53*	33	A-7
19	5	101	71	14	1.9	93*	58	A-7

*WATERLOGGED
SAMPLE NO. 18 - LAYER 4 CONTAINS COARSE GRAVEL SEE DESCRIPTION.

FIGURE 6.—SUBGRADE SURVEY SHEET FOR THE DESIGN OF NEW ROAD

PROCEDURES FOR TESTING SOILS FOR THE DETERMINATION OF THE SUBGRADE SOIL CONSTANTS [a]

By A. M. WINTERMYER, Assistant Highway Engineer, E. A. WILLIS, Assistant Highway Engineer and R. C. THOREEN, Junior Highway Engineer, Bureau of Public Roads

THIS is the fourth of a series of articles on the subject of subgrade soils. The reports published in the June and July, 1931, issues of PUBLIC ROADS (pp. 1 to 49) discussed the soil test constants, their significance, and their application in practice. The report published in the September issue (pp. 51 to 64) described the procedure for making subgrade soil surveys in the field. The purpose of the present report is to acquaint the reader with the procedure employed in testing soils in the subgrade laboratory of the bureau at Arlington, Va.

PREPARATION OF SAMPLE

1. *Apparatus.*—The apparatus consists of the following:

A balance sensitive to 0.1 gram.

A mortar and rubber-covered pestle suitable for breaking up the aggregations of soil particles.

A series of sieves, of square-mesh wire cloth, conforming to the requirements of the standard specifications for sieves for testing purposes of the American Society for Testing Materials (serial designation E–11). The sizes required are shown in Table 1.

A riffle sampler or sample splitter, for quartering the samples.

TABLE 1.—*Requirements for sieve openings and wire diameters with permissible variations*

Mesh designation, U. S. standard sieve series	Sieve opening		Wire diameter		Tolerance			
					Average opening	Wire diameter		Maximum opening
						Under	Over	
No.	*Millimeters*	*Inches*	*Millimeters*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
4	4.76	0.1870	1.27	0.050	±3	15	30	10
10	2.00	.0787	.76	.0299	±3	15	30	10
40	.42	.0165	.25	.0098	±5	15	30	25
200	.074	.0029	.053	.0021	±8	15	35	60

2. *Sample.*—The soil sample as received from the field is dried thoroughly in the air. A representative test sample of the amount required to perform the desired tests is then selected by the method of quartering or by the use of a sampler. The amounts of material required to perform the individual tests are as follows:

(*a*) For the mechanical analysis, material passing No. 10 sieve is required in amounts equal to 115 grams of sandy soils and 65 grams of either silt or clay soils.

(*b*) For the physical tests, material passing the No. 40 sieve is required in total amount equal to 200 grams, allocated as follows:

	Grams
Liquid limit	30
Plastic limit	15
Centrifuge moisture equivalent	10
Field moisture equivalent	50
Volumetric shrinkage	30
Flocculation and check tests	65

(*c*) Physical tests of binder material are performed only when it is desired to investigate the properties of the active constituents in sand-clay and gravel road-surfacing materials. In this case 100 grams of the material passing the No. 200 sieve are required and the tests are performed in the following order: Field moisture equivalent, plastic limit, liquid limit, volumetric shrinkage, and centrifuge moisture equivalent. The material remaining after the conclusion of each test is used in the next test, except that the test for the centrifuge moisture equivalent is made with material not used in previous tests.

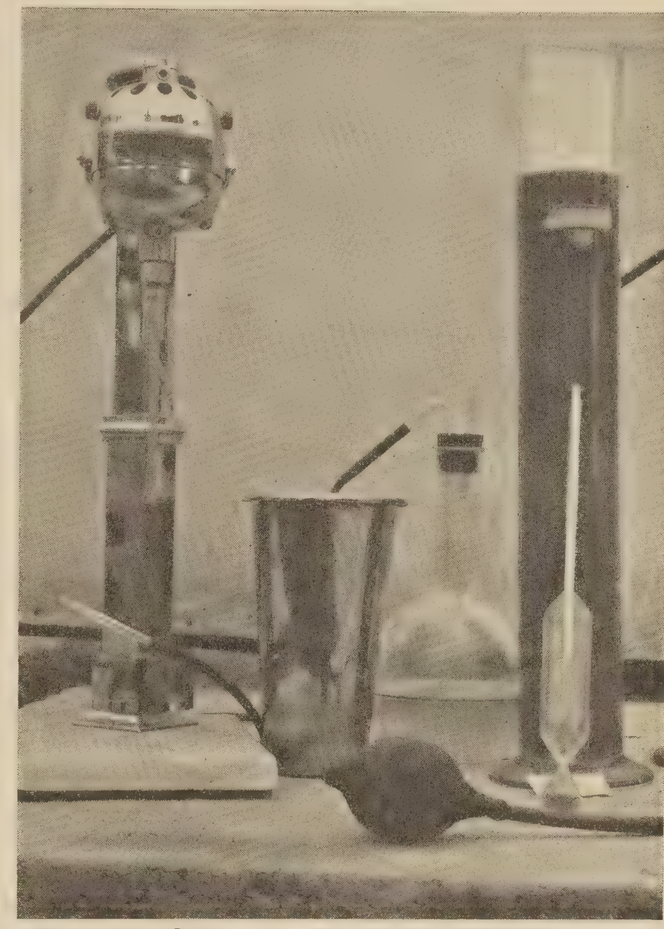


FIGURE 1.—BOUYOUCOS HYDROMETER, CYLINDER, AND SPECIAL MILK-SHAKE MACHINE

3. *Procedure.*—That portion of the air-dried sample selected for test is weighed and the weight recorded as the weight of the total test sample uncorrected for hygroscopic moisture. The test sample is separated by sieving with a No. 10 sieve. That fraction retained on the No. 10 sieve is ground in a mortar with a rubber-covered pestle until the aggregations of soil particles are broken up into the separate grains. The ground soil is then separated into two fractions by sieving with a No. 10 sieve.

That fraction retained after the second sieving is washed free of all fine material, dried, and weighed. This weight is recorded as the weight of coarse material. The coarse material after being washed and dried is sieved on the No. 4 sieve and the weight retained on the No. 4 sieve is recorded.

The fractions passing the No. 10 sieve in both sieving operations are thoroughly mixed together, and by the method of quartering or the use of a sampler a portion weighing approximately 115 grams for sandy soils and approximately 65 grams for silt and clay soils is selected for mechanical analysis.

[a] Reprinted from PUBLIC ROADS, vol. 12, No. 8, October, 1931.

The remaining portion of the material passing the No. 10 sieve is then separated into two parts by means of a No. 40 sieve. The fraction retained on the No. 40 sieve is discarded. The fraction passing the No. 40 sieve is used for the physical tests.

MECHANICAL ANALYSIS BY COMBINED SIEVE AND HYDROMETER METHOD

1. *Apparatus.*—The apparatus consists of the following:

An analytical balance sensitive to 0.001 gram.

A special milk-shake machine with specially designed dispersion cup.

A hydrometer graduated to read grams of soil per liter of suspension. The milk-shake machine and hydrometer, illustrated in Figure 1, were designed by G. J. Bouyoucos and are described in Soil Science, (vol. 23, No. 4, April, 1927, pp. 319 to 330).

A glass graduate 18 inches high and 2½ inches in diameter and graduated for a volume of 1,000 cubic centimeters.

A Fahrenheit thermometer accurate to 1°.

A series of sieves, of square-mesh wire cloth, conforming to the requirements of the standard specifications for sieves for testing purposes of the American Society for Testing Materials (serial designation E–11). The sieves required are shown in Table 2.

A water bath for maintaining the soil suspension at a constant temperature during the hydrometer analysis. This is an insulated zinc tank and maintains the temperature of the suspension at faucet-water temperature. It is illustrated in Figure 2.

A glass cylinder 9½ inches high and 2 inches in diameter, having a capacity of about 425 cubic centimeters.

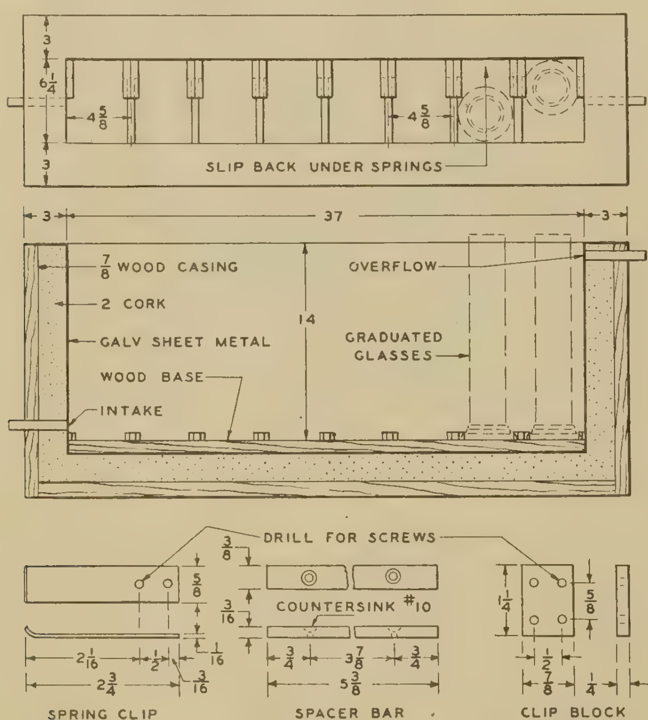


FIGURE 2.—TANK FOR GRADUATED GLASSES, USED FOR MAINTAINING SOIL SUSPENSIONS AT CONSTANT TEMPERATURE DURING HYDROMETER ANALYSIS

2. *Sample.*—Of that portion of the total sample selected for the mechanical analysis, 15 grams are used to determine the hygroscopic moisture and the remainder is used for the combined sieve and hydrometer analysis.

3. *Determination of hygroscopic moisture.*—The 15 grams selected for this purpose are dried to constant weight in an oven at 110° C., weighed, and the results recorded.

4. *Flocculation test.*—Five cubic centimeters of soil particles (weight in grams equal to five times the specific gravity of the soil particles) of that fraction passing the No. 40 sieve are placed in a graduate and

TABLE 2.—*Requirements for sieve openings and wire diameters with permissible variations*

Mesh designation, U. S. standard sieve series	Sieve opening		Wire diameter		Tolerance			
					Average opening	Wire diameter: Under	Wire diameter: Over	Maximum opening
No.	*Millimeters*	*Inches*	*Millimeters*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
20	0.84	0.0331	0.42	0.0165	±5	15	30	25
40	.42	.0165	.25	.0098	±5	15	30	25
60	.25	.0098	.162	.0064	±6	15	35	40
140	.105	.0041	.074	.0029	±8	15	35	60
200	.074	.0029	.053	.0021	±8	15	35	60

vigorously shaken with 45 cubic centimeters of distilled water for two minutes. The graduate is then set aside for a period of 24 hours. If during this period there is evidence of flocculation, the fact is recorded, together with a note regarding the extent of flocculation and the approximate time at which it became evident.

5. *Hydrometer test procedure.*—The portion of air-dried soil selected for mechanical analysis is dispersed by one of the three methods described below. The method to be used is determined by the plasticity index of the soil.

A. In the case of soils having a plasticity index between 0 and 5, the soil is placed in the special dispersion cup and distilled water is added until the cup is within 2 inches of being full. A deflocculating agent, 20 cubic centimeters of sodium silicate solution (3° Baumé at 76° F.), is then added and the contents of the cup are mixed by the special milk-shake machine for a period of five minutes.

B. In the case of soils having a plasticity index between 5 and 20 the soil is placed in a small evaporating dish and completely covered with water. It is allowed to soften under water for a period of at least 18 hours. After the soil has softened it is washed into the special dispersion cup and dispersed in the same manner as in method *A*, except that the time of dispersion is increased to 10 minutes.

C. In the case of soils having a plasticity index greater than 20 the soil is placed in a glass cylinder and to this is added 100 cubic centimeters of 6 per cent hydrogen peroxide. The cylinder is shaken until the soil is completely wetted. The cylinder is then covered with a watch glass and placed in an oven at a temperature of 110° C. for 1 hour, after which it is removed from the oven and allowed to stand for at least 18 hours. The peroxide is used to assist in the dispersion rather than to remove the organic matter. After the soil has been treated with peroxide as described above it is washed into the special dispersion cup and dispersed in the same manner as in method *A*, except that the time of dispersion is increased to 15 minutes.

It is important in all cases to see that the paddle on the dispersion machine is replaced as soon as it shows signs of wear.

After dispersion the mixture is transferred to the glass graduate, and distilled water having the same temperature as the constant temperature bath is added until the mixture attains a volume of 1,000 cubic centimeters. The graduate containing the soil suspension is then placed in the constant temperature bath. The suspension is stirred frequently with a glass rod to prevent settlement of particles in suspension. When the soil suspension attains the temperature of the bath

the graduate is removed and its contents thoroughly shaken for one minute, the palm of the hand being used as a stopper over the mouth of the graduate. At the conclusion of this shaking the time is recorded, the graduate placed in the bath, and readings taken with the hydrometer at the end of both one and two minutes. The hydrometer is read at the top of the meniscus formed by the suspension around its stem to the nearest one-half gram per liter. Subsequent readings are taken at intervals of 5, 15, 30, 60, 250, and 1,440 minutes after the beginning of sedimentation. Readings on the thermometer placed in the constant temperature bath are made coincidentally with the hydrometer readings and recorded.

After each reading except the 1-minute reading, the hydrometer is very carefully removed from the soil suspension in such a manner as to cause no disturbance in the suspension, wiped clean, and laid aside. Fifteen or twenty seconds before the time for a reading it is again slowly and carefully placed in the soil suspension. This operation prevents soil particles from accumulating on the hydrometer and also prevents the hydrometer from reducing the horizontal sectional area of the suspension through which the soil particles settle. The reading is not taken until the hydrometer has come to rest.

6. *Sieve analysis.*—At the conclusion of the final reading the suspension is washed on a No. 200 sieve. That fraction retained on the No. 200 sieve is dried and then analyzed in a nest of sieves consisting of one each of the following: Nos. 20, 40, 60, and 140.

COMPUTATION OF DATA GIVEN BY MECHANICAL ANALYSIS

The data obtained from the three parts of the test, i. e. the separation of coarse material in the preparation of the sample, the hydrometer analysis, and the sieve analysis—are computed and combined as described below and illustrated in Tables 3 and 4.

1. *Hygroscopic moisture.*—The hygroscopic moisture is expressed as a percentage of the weight of the oven-dried soil and is one hundred times the quantity obtained by dividing the difference between the weight of the air-dried and the weight of the oven-dried soil by the weight of the oven-dried soil. The method of computation is illustrated in Table 3.

To correct the weight of the air-dried sample for hygroscopic moisture the given value is multiplied by the expression

$$\frac{100}{100+\text{per cent of hygroscopic moisture}}$$

This factor, as illustrated in Table 3, equals

$$\frac{100}{100+2.53}=0.975$$

Thus in Table 4 the weight of air-dried sample, 99.0 grams, multiplied by 0.975 equals 96.5 grams, the weight of dry soil dispersed.

2. *Coarse material.*—From the weight of the air-dried total test sample (318.3 grams, Table 3) the weight of the oven-dried fraction retained on the No. 10 sieve (56.2 grams, Table 3) is subtracted. The difference (262.1 grams, Table 3) is assumed to equal the weight of the air-dried fraction passing the No. 10 sieve. According to this assumption, no hygroscopic moisture is contained in the air-dried particles retained on the No. 10 sieve, when as a matter of fact a small

TABLE 3.—*Hygroscopic moisture and coarse material determinations for sample 4,422X*

HYGROSCOPIC MOISTURE	
Weight of air-dried soil, grams	15. 00
Weight of dish and air-dried soil, grams	41. 37
Weight of dish and oven-dried soil, grams	41. 00
Loss in weight, grams	0. 37
Weight of oven-dried soil, grams	14. 63
Hygroscopic moisture, percentage of weight of oven-dried soil	2. 53
Hygroscopic moisture correction factor	0. 975
COARSE MATERIAL	
Weight of total test sample, air-dried, grams	318. 3
Weight of washed and oven-dried fraction retained on No. 10 sieve, grams	56. 2
Weight of fraction passing No.10 sieve, air-dried, grams	262. 1
Weight of fraction passing No. 10 sieve corrected for hygroscopic moisture, grams	255. 5
Weight of total test sample corrected for hygroscopic moisture, grams	311. 7
Weight of fraction retained on No. 4 sieve, oven-dried, grams	40. 6
Fraction retained on No. 4 sieve, percentage of corrected weight of total test sample	13. 0
Fraction retained on No. 10 sieve, percentage of corrected weight of total test sample	18. 0

percentage of moisture may be present in this fraction. This amount of moisture, compared with that held in the pores of the fraction passing the No. 10 sieve, is relatively small. Therefore any error produced by the assumption as stated is considered negligible in amount. The weight of the fraction passing the No. 10 sieve is corrected for hygroscopic moisture (255.5 grams, Table 3). To this value is added the weight of the oven-dried fraction retained on the No. 10 sieve to obtain the weight of the total test sample corrected for hygroscopic moisture (311.7 grams, Table 3). The fractions retained on both the No. 4 and the No. 10 sieve are expressed as percentages of the corrected weight of the total test sample (13.0 per cent and 18.0 per cent, respectively, Table 3).

3. *Percentage of soil in suspension.*—For temperatures of the constant temperature bath other than that at which the hydrometer was calibrated, the hydrometer readings are corrected in accordance with temperature correction factors such as are shown graphically as ΔR in Figure 3, A. A temperature correction curve of this type should be determined experimentally for each hydrometer in use. Thus in Table 4 the first hydrometer reading, 34, taken at 70° F. becomes 34.4 when corrected for temperature in accordance with Figure 3, A.

The percentage of the dispersed soil in suspension represented by different corrected hydrometer readings depends upon both the amount and the specific gravity of the soil dispersed.

If the specific gravity of the soil is 2.65, the hydrometer reading gives the weight of soil remaining in suspension in grams per liter of the mixture or suspension. The percentage of dispersed soil remaining in suspension is given by the expression

$$P=\frac{R}{W}\times 100$$

where P = percentage of originally dispersed soil remaining in suspension.
R = hydrometer reading.
W = weight of soil originally dispersed, in grams per liter of suspension.

If, as is the customary procedure, the volume of the suspension is 1 liter, the term W may be taken as the

total weight of soil originally dispersed, without the qualifying phrase, "per liter of suspension."

If the specific gravity of the soil is other than 2.65, the percentage of originally dispersed soil remaining in suspension is given by the expression

$$P=\frac{Ra}{W}\times 100$$

in which a is a constant depending on the density of the suspension. The value of a, for a specific gravity G and a water density at 67° F. of 0.9984, is given by the equation

$$a=\frac{2.6500-0.9984}{2.6500}\times\frac{G}{G-0.9984}$$

Following are values of a for different values of the specific gravity:

Specific gravity, G	Constant, a
2.95	0.94
2.85	0.96
2.75	0.98
2.65	1.00
2.55	1.02
2.45	1.05
2.35	1.08

The value of a for a given specific gravity may be obtained from this table by interpolation. It is sufficiently accurate, however, to select the constant for the specific gravity closest to that of the particular soil tested. Thus, in Table 4, sample No. 4,422X has a specific gravity of 2.41, and consequently the constant, 1.05, corresponding to a specific gravity of 2.45, is used.

A corrected hydrometer reading of 34.4 in Table 4, therefore, indicates a percentage of dispersed soil in suspension,

$$P=\frac{34.4\times 1.05}{96.5\times .01}=37.4 \text{ per cent}$$

For any hydrometer reading R the percentage of dispersed soil in suspension $=R\times 1.088$.

The percentage of the total test sample, including the fraction retained on the No. 10 sieve, is obtained by multiplying this result by the expression

$$\frac{100-\text{per cent retained on the No. 10 sieve}}{100}$$

Thus in Table 4, for a hydrometer reading of 34.4, the percentage of the total sample remaining in suspension is obtained by the computation

$$P_1=37.4\times\frac{100-18.1}{100}=30.7$$

This computation can be combined with the one above, allowing the percentage of the total sample remaining in suspension to be computed from the corrected hydrometer reading R. We have, therefore,

$$P_1=R\times 1.088\times 0.82=0.892\ R$$

Thus for a corrected hydrometer reading of 25.9, Table 4, we obtain the percentage of the total sample in suspension,

$$P_1=25.9\times 0.892=23.1$$

TABLE 4.—*Sieve and hydrometer analysis for sample No. 4,422X*

Percentage of sample retained on No. 10 sieve	18.0
Weight of air-dried sample, grams	99.0
Weight of dry soil dispersed, grams	96.5
Weight of total test sample represented by weight of dry soil, grams	117.7
Specific gravity	2.41
Plasticity index	8.0
Flocculation	None.

DETERMINATION OF PERCENTAGE OF SOIL IN SUSPENSION

Date tested	Time observed	Temperature	Hydrometer reading: Original	Hydrometer reading: Corrected for temperature (fig. 3, A)	Percentage of dispersed sample remaining in suspension, P	Percentage of total test sample remaining in suspension, P_1
		°F.				
July 29, 1929	9.41 a. m.					
Do	9.42 a. m.	70	34.0	34.4	37.4	30.7
Do	9.43 a. m.	70	25.5	25.9	28.2	23.1
Do	9.46 a. m.	70	19.0	19.4	21.1	17.3
Do	9.56 a. m.	70	15.0	15.4	16.8	13.7
Do	10.11 a. m.	70	12.0	12.4	13.5	11.1
Do	10.41 a. m.	70	10.5	10.9	11.9	9.7
Do	1.51 p. m.	70	7.0	7.4	8.1	6.6
July 30, 1929	9.41 a. m.	68	3.0	3.1	3.4	2.8

DETERMINATION OF SIZE OF SOIL PARTICLES IN SUSPENSION

Original hydrometer reading	Period of sedimentation, T	Grain diameter[1] D	Temperature	Correction coefficients: K_L, Fig. 3, B	K_G, Fig. 3, C	K_D, Fig. 3, D	Corrected grain diameter
	Minutes	*Millimeters*	°F.				*Millimeters*
34.0	1	0.078	70	0.48	1.08	0.98	0.0396
25.5	2	.055	70	.50	1.08	.98	.0291
19.0	5	.035	70	.51	1.08	.98	.0189
15.0	15	.020	70	.52	1.08	.98	.0110
12.0	30	.014	70	.53	1.08	.98	.0079
10.5	60	.010	70	.53	1.08	.98	.0056
7.0	250	.005	70	.54	1.08	.98	.0029
3.0	1,440	.002	68	.55	1.08	.99	.0012

[1] See table 5.

SIEVE ANALYSIS

Fraction	Weight	Percentage of total test sample
	Grams	
Passing No. 10, retained on No. 20	2.35	2.0
Passing No. 20, retained on No. 40	2.59	2.2
Passing No. 40, retained on No. 60	4.12	3.5
Passing No. 60, retained on No. 140	9.41	8.0
Passing No. 140, retained on No. 200	12.11	10.3

4. *Diameter of soil particles in suspension.*—The maximum diameters of the particles in suspension, based on Stokes's law for assumed conditions suggested in part by G. J. Bouyoucos, are shown in Table 5. (See Soil Science, vol. 26, No. 3, September, 1928, p. 234.)

According to Stokes's law,

$$d=\sqrt{\frac{30nL}{980\ (G-G_1)\ T}}$$

In this equation—

d = maximum grain diameter in millimeters.

n = coefficient of viscosity of the suspending medium (in this case water) in poises. Varies with changes in temperature of the suspending medium.

L = distance in centimeters through which soil particles settle in a given period of time.

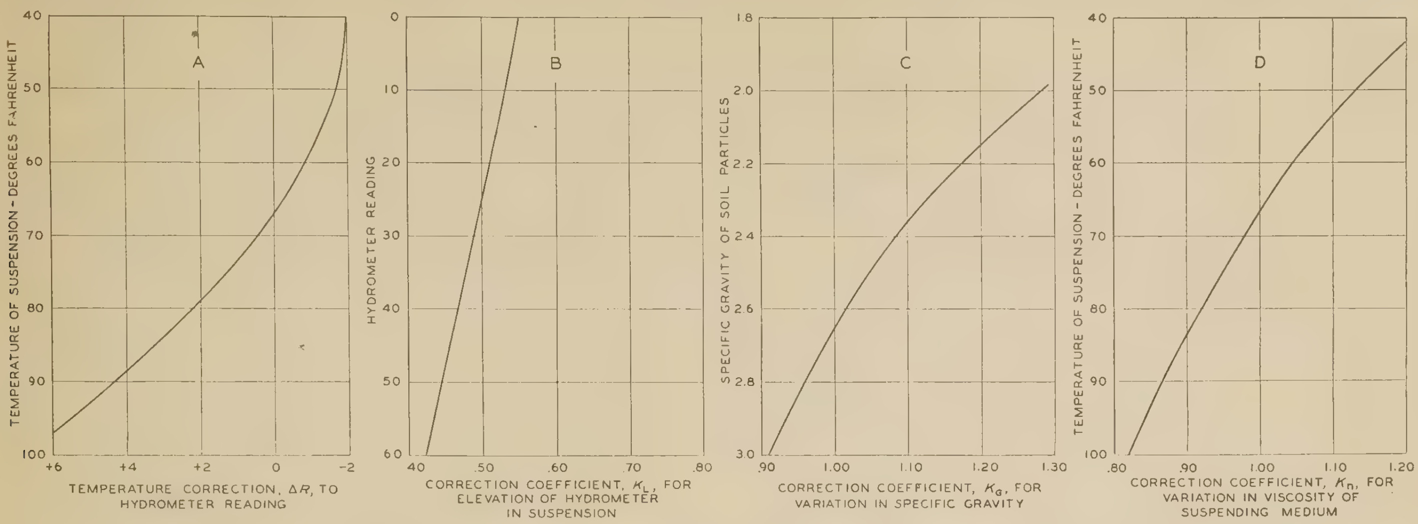


FIGURE 3.—CORRECTION CURVES FOR USE IN HYDROMETER ANALYSIS

T = time in minutes, period of sedimentation.
G = specific gravity of soil particles.
G_1 = specific gravity of the suspending medium. In this case $G_1 = 0.9984$, or approximately 1.0.

TABLE 5.—*Maximum grain diameters in suspension under assumed conditions*

Time	Maximum grain diameter in suspension
Minutes	*Millimeters*
1	0.078
2	.055
5	.035
15	.020
30	.014
60	.010
250	.005
1,440	.002

The grain diameters given in Table 5 are computed according to the following assumptions:

L, the distance through which the particle falls, is constant and equal to 32.5 centimeters.

n, the coefficient of viscosity, equals 0.0102, that of water at 67° F.[1]

G, the specific gravity of the soil particles, is constant and equal to 2.65.

As a matter of fact, the hydrometer reading is dependent, not on particles distributed throughout a depth of 32.5 centimeters in the suspension, but on those existing in that portion of the suspension holding the hydrometer.

In order to use Stokes's law to determine the diameter of the particles it is necessary to know the distance through which these particles fall in a given time. Since the density throughout a suspension is not uniform and varies with the grading of the material in suspension and the time of sedimentation, a fixed distance can not be used. For hydrometers of certain shapes the depth of the center of volume of the hydrometer below the surface of the suspension could be taken as the distance through which the particles may be assumed to fall. In the case of the Bouyoucos hydrometer it has been found by experiment that for the methods of dispersion described in this procedure an assumed distance which bears a constant ratio to the depth of the hydrometer in the suspension, but which is less than the distance indicated by the center of volume of the hydrometer, gives closer agreement to mechanical analysis performed by the pipette methods. The assumed distance of fall has been taken as 0.42 of the distance from the surface of the suspension to the elevation of the bottom of the hydrometer.

The specific gravities of the soil particles and the temperature of the suspension are likely to vary from those assumed in the preparation of Table 5. A better approximation to the true diameters of the soil particles is obtained by applying correction coefficients to the values given by Table 5.

Curves from which these coefficients may be derived are given in Figure 3. The correction coefficients for elevation of hydrometer (fig. 3, B) are obtained experimentally for each hydrometer in use. The coefficients for the specific gravity and the viscosity correction (figs. 3, C and 3, D, respectively) are independent of the apparatus used in the test.

Multiplication by the coefficient shown in Figure 3, B gives the maximum grain size at the reference elevation in the suspension instead of that at a depth of 32.5 centimeters. This coefficient varies with the hydrometer reading and is given by the expression,

$$K_L = \sqrt{\frac{\text{assumed depth of fall in centimeters}}{32.5}}$$

Multiplication by the coefficient shown in Figure 3, C corrects for variation in specific gravity[2] from that on which the sizes given in Table 5 are based and is given by the expression

$$K_G = \sqrt{\frac{1.65}{\text{specific gravity of soil particle} - 1}}$$

Multiplication by the coefficient shown in Figure 3, D corrects for the viscosity of water at temperatures other than 67° F., the temperature of the suspension assumed in the preparation of Table 5. The viscosity correction coefficient is given by the expression

$$K_n = \sqrt{\frac{\text{viscosity coefficient at given temperature}}{0.0102}}$$

The application of these coefficients is illustrated in Table 4. After a period of sedimentation of one minute

[1] Smithsonian Physical Tables, seventh revised edition, 1921, p. 155

[2] The specific gravity of the soil should be obtained by the pycnometer method.

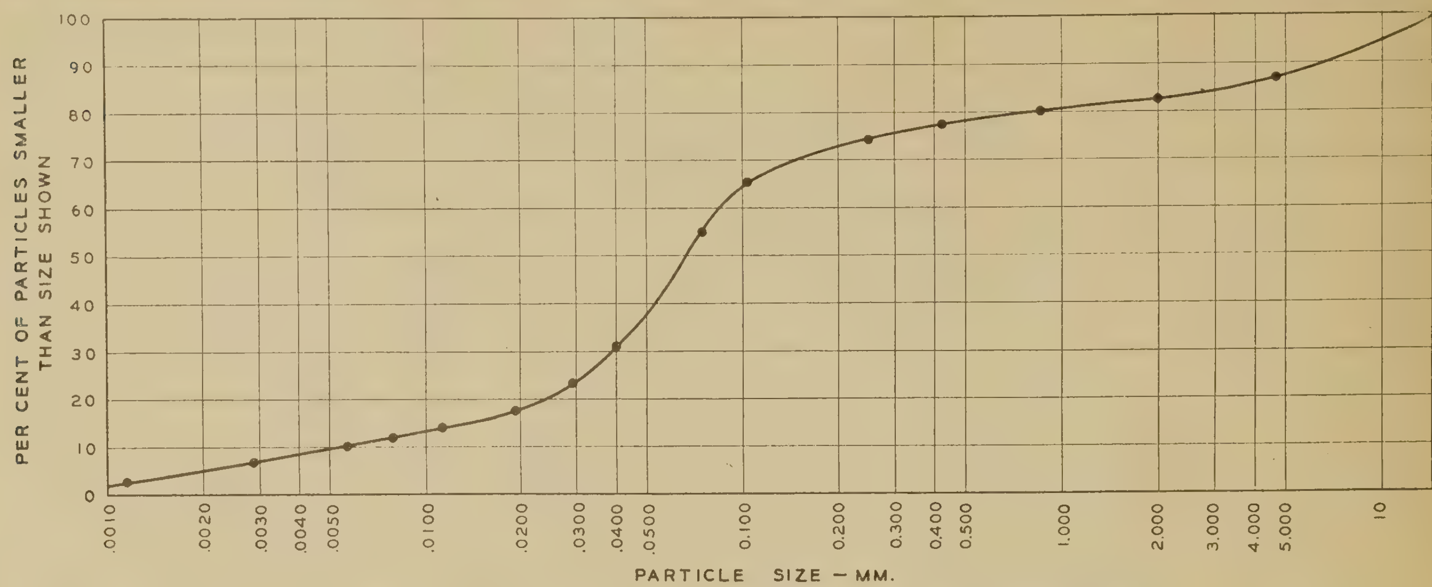


Figure 4.—Grain-Size Accumulation Curve for Soil Sample 4,422X

the grain diameter indicated by Table 5 is 0.078 millimeters. The uncorrected hydrometer reading was 34, the specific gravity was 2.41, and the temperature of the suspension was 70° F. The correction coefficients in this case are 0.48, corresponding to a hydrometer reading of 34 (fig. 3, B); 1.08, corresponding to a specific gravity of 2.41 (fig. 3, C); and 0.98, corresponding to a water temperature of 70° F. (fig. 3, D). The corrected grain diameter then becomes

$$0.078 \text{ millimeter} \times 0.48 \times 1.08 \times 0.98 = 0.040 \text{ millimeter}$$

5. *Sieve analysis.*—The percentage of the soil sample retained on each of the sieves in the sieve analysis is obtained by dividing the weight of fraction retained on each sieve by the weight of the oven-dried fraction dispersed (117.7 grams, Table 4) and multiplying by 100.

6. *Plotting.*—The percentages of grains of different diameters are plotted to a logarithmic scale to obtain "soil grain-diameter accumulation curves." Figure 4 shows a curve of this character representing the mechanical analysis of soil sample No. 4,422X.

7. *Record.*—The results are reported as follows:

Particles larger than 2 millimeters, per cent.
Coarse sand, 2.0 millimeters to 0.25 millimeter, per cent.
Fine sand, 0.25 millimeter to 0.05 millimeter, per cent.
Silt, 0.05 millimeter to 0.005 millimeter, per cent.
Clay, smaller than 0.005 millimeter, per cent.
Colloids, smaller than 0.001 millimeter, per cent.

DETERMINATION OF LIQUID LIMIT

1. *Definition.*—The liquid limit of a soil is that moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil will just begin to flow when lightly jarred ten times.

2. *Apparatus.*—The apparatus consists of the following:

A porcelain evaporating dish about 4½ inches in diameter.
A flexible spatula having a blade about 3 inches long and about ¾ inch wide.
A grooving tool of dimensions shown in Figure 5.
Matched watch glasses which are held together by a suitable clamp and fit sufficiently tight to prevent loss of moisture during weighing.
An analytical balance sensitive to 0.001 gram.

3. *Sample.*—A sample weighing about 30 grams is taken from the thoroughly mixed portion of the material passing the No. 40 sieve.

4. *Procedure.*—The air-dried soil is placed in the evaporating dish and thoroughly mixed with water until the mass becomes pasty. The mass of soil is then shaped into a smooth layer about three-eighths inch thick at the center and divided into two portions with the grooving tool, as shown in Figure 6, top.

The dish is held firmly in one hand, with the groove parallel to the line of sight, and tapped lightly with a

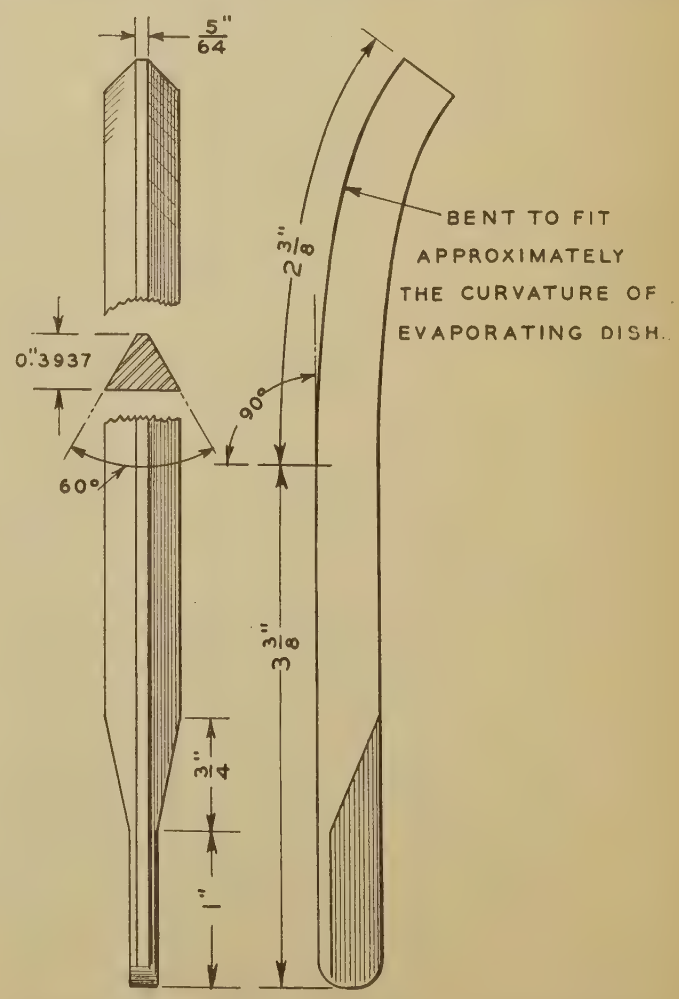
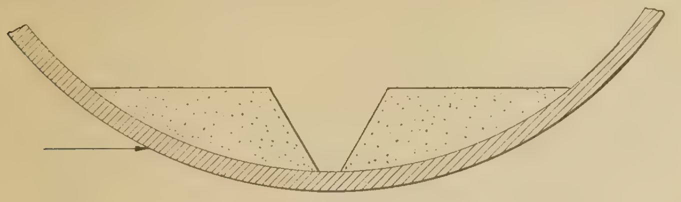
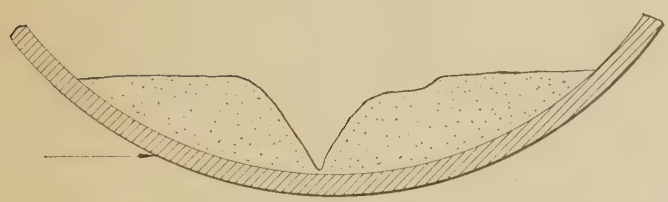


Figure 5.—Grooving Tool Used in Liquid Limit Test



DIVIDED SOIL CAKE BEFORE TEST



SOIL CAKE AFTER TEST

FIGURE 6.—DIAGRAM ILLUSTRATING LIQUID LIMIT TEST

horizontal motion against the palm of the other hand ten times. If the lower edges of the two soil portions do not flow together as shown in Figure 6, bottom, after ten blows have been struck, the moisture content is below the liquid limit. More water should be added and the procedure repeated. If the lower edges meet before ten blows have been struck, the moisture content is above the liquid limit, and dry soil should be added and the procedure repeated.

When the lower edges of the two portions of the soil cake just flow together as shown in Figure 6, bottom, after ten blows have been struck, the moisture content equals the liquid limit. To determine definitely whether the two portions are actually joined, the spatula is used to push one away from the other. If the two portions separate along the original line of division, the end point has not been reached, and the procedure is repeated with the addition of a small amount of water.

A small quantity of soil from that portion of the soil cake which has flowed is removed and placed in a pair of watch glasses. The watch glasses and soil are then weighed and the weight recorded as the weight of glass and wet soil (37.49 grams in Table 6). The soil in the glasses is oven-dried to constant weight at a temperature of 110° C. and weighed. This weight is recorded as the weight of glass and dry soil (28.15 grams, Table 6). The loss in weight due to drying (37.49 grams − 28.15 grams = 9.34 grams, Table 6) is recorded as the weight of water.

5. *Calculation.*—The liquid limit is expressed as the moisture content in percentage of the weight of the oven-dried soil. It is computed from the following formula:

$$L.\ L. = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

Thus in Table 6 the liquid limit of sample S 5,214 is

$$\frac{9.34}{15.06} \times 100 = 62.0 \text{ per cent}$$

DETERMINATION OF PLASTIC LIMIT

1. *Definition.*—The plastic limit of a soil is the lowest moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil can be rolled into threads one-eighth inch in diameter without the threads breaking into pieces.

TABLE 6.—*Plasticity determinations for sample S 5,214*

LIQUID LIMIT TEST	
Weight of glass and wet soil, grams	37.49
Weight of glass and dry soil, grams	28.15
Weight of glass, grams	13.09
Weight of water, grams	9.34
Weight of dry soil, grams	15.06
Liquid limit, per cent	62.0
PLASTIC LIMIT TEST	
Weight of glass and wet soil, grams	32.20
Weight of glass and dry soil, grams	28.59
Weight of glass, grams	12.17
Weight of water, grams	3.61
Weight of dry soil, grams	16.42
Plastic limit, per cent	22.0
Plasticity index, per cent	40.0

FIGURE 7.—ROLLING OF SOIL THREADS IN PLASTIC LIMIT TEST

2. *Apparatus.*—The apparatus consists of the following:

A porcelain evaporating dish about 4½ inches in diameter.

A flexible spatula having a blade about 3 inches long and about three-fourths inch wide.

A glass plate or piece of glazed paper on which to roll the sample.

Matched watch glasses which are held together by a suitable clamp and fit sufficiently tight to prevent loss of moisture during weighing.

An analytical balance sensitive to 0.001 gram.

3. *Sample.*—A sample weighing about 15. grams is taken from the thoroughly mixed portion of the material passing the No. 40 sieve.

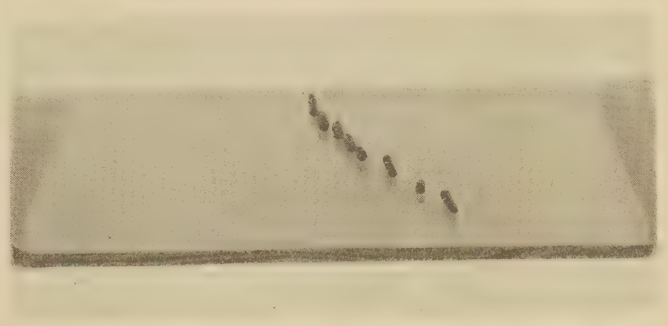


FIGURE 8.—CRUMBLED SOIL THREADS RESULTING FROM PLASTIC LIMIT TEST

4. *Procedure.*—The air-dried soil is placed in the evaporating dish and mixed with water until the mass becomes plastic enough to be easily shaped into a ball. The ball of soil is then rolled between the palm of the hand and the glass plate or piece of glazed paper with just sufficient pressure to form the soil mass into a thread. (Fig. 7.) When the diameter of the resulting thread becomes one-eighth of an inch the soil is kneaded together and again rolled out. This process is continued until the crumbling of the soil (as shown in fig. 8) prevents the formation of the thread. The portions of

the crumbled soil are then gathered together and placed in watch glasses. The watch glasses and soil are weighed and the weight recorded as the weight of glass and wet soil (32.20 grams, Table 6). The soil in the glasses is then oven-dried to constant weight at a temperature of 110° C. and weighed. This weight is recorded as the weight of glass and dry soil (28.59 grams, Table 6). The loss in weight (32.20 grams − 28.59 grams = 3.61 grams, Table 6) is recorded as the weight of water.

5. *Calculations.*—The plastic limit is expressed as the moisture content in percentage of the weight of the oven-dry soil. It is computed from the following formula:

$$P.\ L. = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

Thus in Table 6 the plastic limit of sample S 5,214 equals

$$\frac{3.61}{16.42} \times 100 = 22.0 \text{ per cent}$$

DETERMINATION OF PLASTICITY INDEX

1. *Definition.*—The plasticity index of a soil is the difference between its liquid limit and its plastic limit.

2. *Calculation.*—The plasticity index is calculated by the formula $P.\ I. = L.\ L. - P.\ L.$

Thus in Table 6 the plasticity index of sample S 5,214 equals 62.0 − 22.0 = 40.0 per cent.

DETERMINATION OF CENTRIFUGE MOISTURE EQUIVALENT

1. *Definition.*—The centrifuge moisture equivalent of a soil is the amount of moisture, expressed as a percentage of the weight of the oven-dried soil, retained by a soil which has been first saturated with water and then subjected to a force equal to one thousand times the force of gravity for one hour.

2. *Apparatus.*—The apparatus consists of the following:

A porcelain Gooch crucible with perforated bottom. The crucible is about 1½ inches in height and about 1 inch in diameter at the top and three-fourths of an inch at the bottom, outside dimensions.

A circular piece of filter paper just large enough to cover the inside bottom of the Gooch crucible.

A Babcock trunnion cup fitted with a brass cap and with a rubber stopper with a hole in the center, as shown in Figure 9.

A centrifuge of such size and so driven that a force equal to one thousand times the force of gravity may be exerted on the center of gravity of the soil sample.

An analytical balance sensitive to 0.001 gram.

3. *Sample.*—A 5-gram sample is taken from the thoroughly mixed portion of the material passing the No. 40 sieve.

4. *Number of tests.*—Tests are made in duplicate. Table 7 shows the record for the two tests of sample S 5,214.

5. *Procedure.*—The sample is placed in the Gooch crucible, in which has previously been placed a piece of wet filter paper which just covers the bottom of the crucible. The crucible is placed in a pan of water and the sample allowed to take up moisture until completely saturated, as indicated by the presence of free water on the surface of the sample. It is then placed in a humidifier for at least 12 hours to insure uniform distribution of moisture throughout the soil mass. All free water then remaining on the surface of the sample is poured off, and the crucible is placed in a Babcock cup fitted with a rubber stopper, as shown in Figure 9. The hole in the stopper should be large enough to receive the water forced from the soil by the centrifuging operation. In addition to receiving the ejected water the stopper serves also as a cushion for the crucible. The sample is centrifuged for a period of one hour at a speed which, for the diameter of head used, will exert a centrifugal force one thousand times the force of gravity upon the center of gravity of the soil sample. Immediately after centrifuging, the crucible and contents are weighed and the weight recorded as the weight of crucible and contents after centrifuging (15.74 grams, Table 7). The sample is then oven-dried to constant weight at a temperature of 110° C. and weighed. This weight is recorded as the weight of crucible and contents after drying (12.82 grams, Table 7).

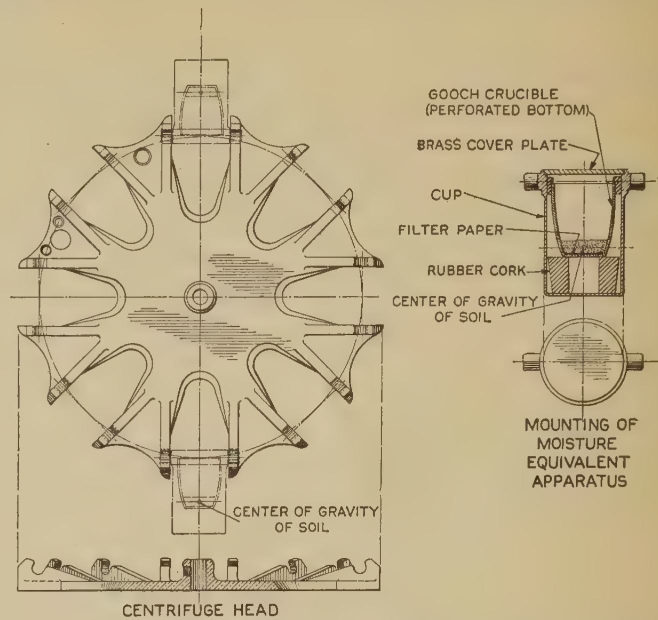


FIGURE 9.—APPARATUS FOR DETERMINING THE CENTRIFUGE MOISTURE EQUIVALENT

6. *Water-logging.*—When free water is observed on the top of the sample after the centrifuging operation the soil is said to have water-logged. This water is not removed, but is weighed with the sample.

7. *Calculation.*—The centrifuge moisture equivalent of the soil is calculated from the formula,

$$C.\ M.\ E. = \frac{(A-b)-(A_1-b_1)}{A_1-(c+b_1)} \times 100$$

in which

A = weight of crucible and contents after centrifuging.
A_1 = weight of crucible and contents after drying.
c = weight of crucible.
b = weight of filter paper wet.
b_1 = weight of filter paper dry.

Thus in Table 7 the centrifuge moisture equivalent of sample S 5,214, as given by the first test, is obtained by the computation,

$$C.\ M.\ E. = \frac{(15.74-0.20)-(12.82-0.10)}{12.82-(7.86+0.10)} \times 100$$

$$= \frac{2.82}{4.86} \times 100 = 58.0 \text{ per cent.}$$

8. *Variation.*—The variation between the two values obtained should not exceed 1 per cent for values of the moisture equivalent up to 15 and 2 per cent for values above 15.

TABLE 7.—*Determination of centrifuge moisture equivalent for sample S 5,214*

Test No.	Weight [1] of— Crucible and contents after centrifuging	Crucible and contents after drying	Crucible	Centrifuge moisture equivalent
	Grams	*Grams*	*Grams*	*Per cent*
1	15.74	12.82	7.86	[2] 58.0
2	16.03	13.00	8.02	[2] 60.0
Average				59.0

[1] Weight of filter paper: Wet, b=0.20 gram; dry, b_1=0.10 gram.
[2] Water-logged.

DETERMINATION OF FIELD MOISTURE EQUIVALENT

1. *Definition.*—The field moisture equivalent of a soil is defined as the minimum moisture content, expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed by the soil but will spread out over the surface and give it a shiny appearance.

2. *Apparatus.*—The apparatus consists of the following:

A porcelain evaporating dish about 4½ inches in diameter.
A flexible spatula having a blade about 3 inches long and about three-fourths inch wide.
A pipette, burette, or similar device for adding water dropwise.
Matched watch glasses, held together by a suitable clamp and fitting sufficiently tight to prevent loss of moisture during weighing.
An analytical balance sensitive to 0.001 gram.

3. *Sample.*—A sample weighing about 50 grams is taken from the thoroughly mixed portion of the material passing the No. 40 sieve.

4. *Procedure.*—The air-dried sample is placed in the evaporating dish and mixed with water. Water is added in small amounts and the sample is thoroughly mixed after each addition of water. When the wetted soil forms into balls under manipulation the sample is smoothed off with a light stroke of the spatula and a drop of water is placed on the smoothed surface. If the water immediately disappears a few more drops of water are added, and the procedure is repeated until the water does not immediately disappear but spreads over the smoothed surface and leaves a shiny appearance. A small portion of the soil on which the last drop was placed is then removed and placed between two watch glasses. The weight of the watch glasses and wet soil is determined and recorded (32.08 grams, Table 8). The sample is then oven-dried to constant weight at a temperature of 110° C. and weighed. This weight is recorded as the weight of glass and dry soil (26.29 grams, Table 8). The difference in weight (32.08 grams − 26.29 grams = 5.79 grams, Table 8) is recorded as the weight of water.

5. *Calculations.*—The results obtained in the determination of the field moisture equivalent of sample S 5,214 are given in Table 8. The field moisture equivalent is computed by means of the formula

$$F.\ M.\ E. = \frac{\text{weight of water}}{\text{weight of oven-dried soil}} \times 100$$

Thus in Table 8 the field moisture equivalent of sample S 5,214 equals

$$\frac{5.79}{14.12} \times 100 = 41.0 \text{ per cent.}$$

TABLE 8.—*Determination of field moisture equivalent of sample S 5,214*

Weight of glass and wet soil, grams	32.08
Weight of glass and dry soil, grams	26.29
Weight of glass, grams	12.17
Weight of water, grams	5.79
Weight of dry soil, grams	14.12
Field moisture equivalent, per cent	41.0

SHRINKAGE DETERMINATION

1. *Scope.*—This procedure furnishes the data from which the following subgrade soil constants may be computed: (*a*) Shrinkage limit, (*b*) shrinkage ratio, (*c*) volumetric change, (*d*) lineal shrinkage, and (*e*) specific gravity (approximate). Shrinkage determinations made on sample S 5,214 are recorded in Table 9.

2. *Apparatus.*—The apparatus consists of the following:

A porcelain evaporating dish about 4½ inches in diameter.
A flexible spatula having a blade about 3 inches long and about three-fourths inch wide.
A circular porcelain milk dish having a flat bottom and being about 1¾ inches in diameter by about one-half inch high.
A steel straightedge about 12 inches long.
A glass cup about 2 inches in diameter and about 1 inch high, the top rim of which is ground smooth and level.
A glass plate with three metal prongs for immersing the soil pat in mercury, as shown in Figure 10.
A glass graduate having a capacity of 25 cubic centimeters and graduated to 0.2 cubic centimeter.
An analytical balance sensitive to 0.001 gram.
Sufficient mercury to fill the glass cup to overflowing.

3. *Sample.*—A sample weighing about 30 grams is taken from the thoroughly mixed portion of the material passing the No. 40 sieve.

4. *Procedure.*—The sample is placed in the evaporating dish and thoroughly mixed with water in amount sufficient to fill the soil voids completely and to make the soil pasty enough to be readily worked into the porcelain milk dish without the inclusion of air bubbles. The amount of water required to furnish friable soils with the desired consistency is equal to or slightly greater than the liquid limit, and the amount necessary to furnish plastic soils with the desired consistency may exceed the liquid limit by as much as 10 per cent. The inside of the porcelain milk dish is coated with a thin layer of vaseline or some other heavy grease to prevent the adhesion of the soil to the dish.

An amount of the wetted soil equal to about one-third the volume of the milk dish is placed in the center of the dish, and the soil is caused to flow to the edges by tapping the dish on a firm surface cushioned by several layers of blotting paper or similar material. An amount of soil is added approximately equal to the first portion, and the dish is tapped until the soil is thoroughly compacted and all included air is brought to the surface. More soil is added and the tapping is continued until the dish is completely filled and excess soil stands out about its edge. The excess soil is then struck off with a straightedge, and all soil adhering to the outside of the dish is wiped off.

The dish when filled and struck off is weighed immediately and the weight recorded as the weight of dish and wet soil (29.34 grams, Table 9). The soil pat is allowed to dry in air until the color of the pat turns from dark to light. It is then oven-dried to constant weight at 110° C. and the weight recorded as the weight of dish and dry soil (22.61 grams, Table 9). The weight of the empty dish (11.52 grams, Table 9) is determined and recorded. The capacity of the dish in cubic centimeters, which is also the

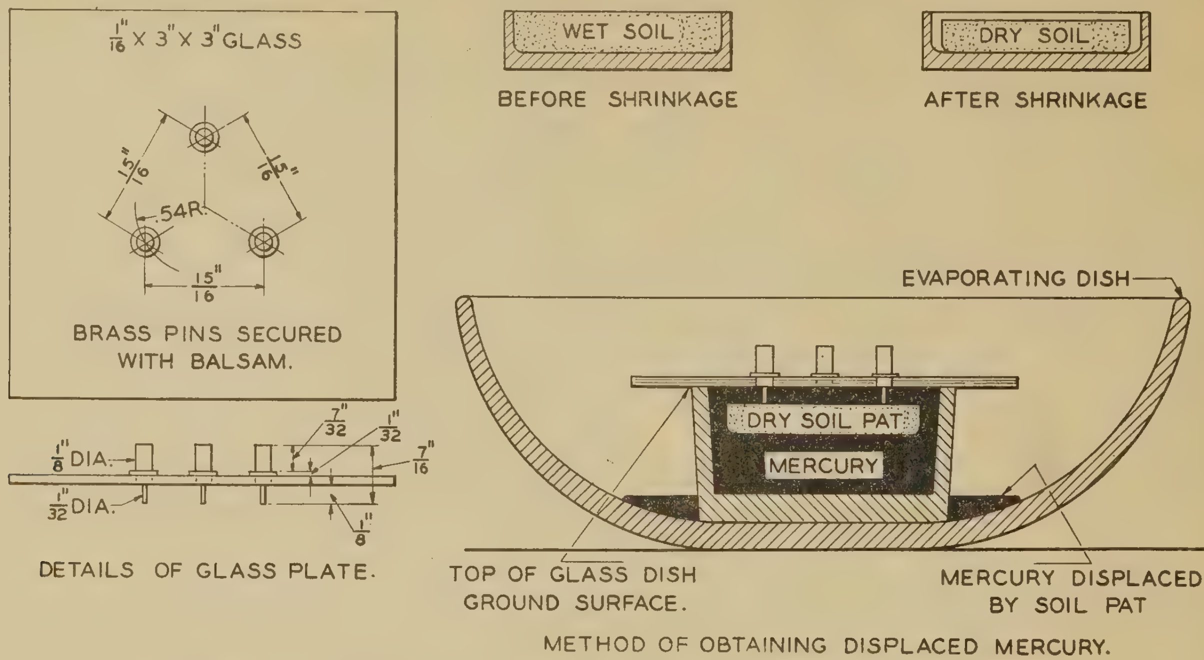


Figure 10.—Apparatus for Determining the Volumetric Change

volume of the wet soil pat, is determined by filling the dish to overflowing with mercury, removing the excess by pressing a glass plate firmly over the top of the dish, and measuring the volume of mercury held in the dish in the glass graduate. This volume is recorded as the volume of the wet soil pat, V (10.99 cubic centimeters, Table 9).

The volume of the dry soil pat is determined by removing the pat from the porcelain milk dish and immersing it in the glass cup full of mercury in the following manner: The glass cup is filled to overflowing with mercury and the excess mercury is removed by pressing the glass plate with the three prongs (fig. 10) firmly over the top of the cup. Any mercury which may be adhering to the outside of the cup is carefully wiped off. The cup, filled with mercury, is placed in the evaporating dish, and the soil pat is placed on the surface of the mercury. It is then carefully forced under the mercury by means of the glass plate with the three prongs (fig. 10) and the plate is pressed firmly over the top of the cup. It is essential that no air be trapped under the soil pat. The volume of the mercury so displaced is measured in the glass graduate and recorded as the volume of the dry soil pat, V_o (5.60 cubic centimeters, Table 9).

5. *Computations.*—The weight of the milk dish is subtracted from the weight of dish and wet pat to give the weight of the wet soil pat, W. The weight of the milk dish subtracted from the weight of dish and dry pat gives the weight of the dry soil pat, W_o The moisture content w of the soil at the time it was put in the dish, expressed as a percentage of the dry weight of the soil, is computed from the formula

$$w=\frac{W-W_o}{W_o}\times 100$$

Thus in Table 9

$$W=29.34-11.52=17.82 \text{ grams},$$

$$W_o=22.61-11.52=11.09 \text{ grams},$$

and

$$w=\frac{17.82-11.09}{11.09}\times 100=60.7 \text{ per cent.}$$

CALCULATION OF SHRINKAGE LIMIT

1. *Definition.*—The shrinkage limit of a soil is that moisture content, expressed as a percentage of the weight of the oven-dried soil, at which a reduction in moisture content will not cause a decrease in the volume of the soil mass, but at which an increase in moisture content will cause an increase in the volume of the soil mass.

2. *Computations.*—The shrinkage limit, S, is calculated from the data obtained in the volumetric shrinkage determination by the following formula:[3]

$$S=w-\left(\frac{V-V_o}{W_o}\times 100\right)$$

Thus in Table 9 the shrinkage limit of sample S 5,214 equals

$$60.7-\left(\frac{10.99-5.60}{11.09}\times 100\right)=12.1 \text{ per cent}$$

3. *Optional method.*—When both the true specific gravity, G, and the shrinkage ratio, R, are known, the shrinkage limit may be calculated from the formula:[3]

$$S=\left(\frac{1}{R}-\frac{1}{G}\right)\times 100$$

Thus in Table 9, if G and R were known first, the shrinkage limit of sample S 5,214 could be computed as follows:

$$S=\left(\frac{1}{1.98}-\frac{1}{2.60}\right)\times 100=12.1 \text{ per cent}$$

CALCULATION OF SHRINKAGE RATIO

1. *Definition.*—The shrinkage ratio of a soil is the ratio between a given volume change, expressed as a percentage of the dry volume, and the corresponding change in moisture content above the shrinkage limit, expressed as a percentage of the weight of the oven-dried soil. It equals the apparent specific gravity of the dried soil pat.

[3] See p. 29.

2. *Computations.*—The shrinkage ratio, R, is calculated from the data obtained in the volumetric shrinkage determination by the following formula:[3]

$$R=\frac{W_o}{V_o}$$

Thus in Table 9 the shrinkage ratio of sample S 5,214 equals

$$\frac{11.09}{5.60}=1.98$$

CALCULATION OF VOLUMETRIC CHANGE

1. *Definition.*—The volumetric change of a soil for a given moisture content is the volume change, expressed as a percentage of the dry volume, suffered by the soil mass when the moisture content is reduced from the stipulated percentage to the shrinkage limit. This stipulated moisture content is usually taken as the field moisture equivalent.

2. *Computation.*—The volumetric change, $V.\ C.$, is calculated from the data obtained in the volumetric shrinkage determination by the following formula:

$$V.\ C.=(w_1-S)R$$

where w_1 is the given moisture content.

If, as is customary, the volumetric change from the field moisture equivalent is desired, the formula assumes the form,

C_f = volumetric change from field moisture equivalent[4] $=(F.\ M.\ E.\ -S)R.$

Thus in Table 9 the volumetric change of sample S 5,214, when mixed with an amount of water equal to the field moisture equivalent (see Table 8) equals $(41.0-12.1)\times 1.98=57.2$ per cent.

CALCULATION OF LINEAL SHRINKAGE

1. *Definition.*—The lineal shrinkage of a soil for a given moisture content is the decrease in one dimension, expressed as a percentage of the original dimension, suffered by the soil mass when the moisture content is reduced from an amount equal to the field moisture equivalent to the shrinkage limit.

2. *Computation.*—The lineal shrinkage, $L.\ S.$, is obtained either by means of the formula[4]

$$L.\ S.=100\left(1-\sqrt[3]{\frac{100}{C_f+100}}\right)$$

or by means of the curve shown in Figure 11, which represents this relation. Thus in Table 9 the lineal shrinkage of sample S 5,214, as obtained from the formula, is 14.0 per cent.

TABLE 9.—*Volumetric shrinkage determination for sample S 5,214*

Weight of dish and wet soil, grams	29.34
Weight of dish and dry soil, grams	22.61
Weight of dish, grams	11.52
W, weight of wet soil pat, grams	17.82
W_o, weight of dry soil pat, grams	11.09
w, moisture content of wet soil pat, per cent	60.7
V, volume of dish, volume of wet soil pat, cubic centimeters	10.99
V_o, volume of dry soil pat, cubic centimeters	5.60
S, shrinkage limit, per cent	12.1
R, shrinkage ratio	1.98
C_f, volumetric change from field moisture equivalent, per cent	57.2
$L.\ S.$, lineal shrinkage, per cent	14.0
G, specific gravity (approximate)	2.60

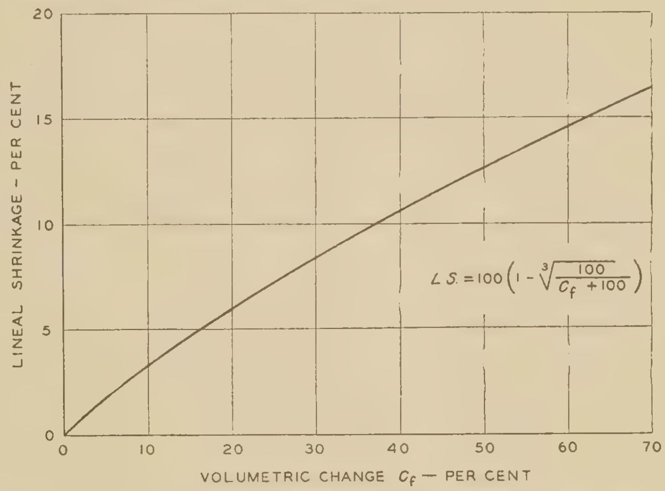


FIGURE 11.—RELATION BETWEEN VOLUMETRIC CHANGE AND LINEAL SHRINKAGE

APPROXIMATE CALCULATION OF SPECIFIC GRAVITY

1. *Definition.*—The specific gravity of a soil is the weight of the oven-dried soil divided by the true volume of the soil particles.

2. *Computations.*—The specific gravity, G, is calculated from the data obtained in the volumetric shrinkage test by the following formula:[4]

$$G=\frac{1}{\frac{1}{R}-\frac{S}{100}}$$

Thus in Table 9 the approximate specific gravity of sample S 5,214 is obtained by the computation

$$G=\frac{1}{\frac{1}{1.98}-\frac{12.1}{100}}=\frac{1}{0.384}=2.60$$

[3] See p. 29.

[4] See pp. 34 and 35.

GRAPHICAL SOLUTION OF THE DATA FURNISHED BY THE HYDROMETER METHOD OF ANALYSIS [a]

Reported by E. A. WILLIS, Assistant Highway Engineer, F. A. ROBESON, Junior Highway Engineer, and C. M. JOHNSTON, Junior Civil Engineer, Bureau of Public Roads

THE HYDROMETER method of mechanical analysis described previously in this issue of PUBLIC ROADS includes the following two separate operations: (*a*) The determination of the percentage of dispersed soil particles remaining in suspension at a given time, as indicated by particular hydrometer readings, and (*b*) the determination of the maximum size of soil particles in suspension corresponding to the percentages represented by particular hydrometer readings. The percentage of particles in suspension is determined by the hydrometer reading corrected for the conditions under which the test is performed. The maximum grain size at the time of any particular hydrometer reading is computed by means of Stokes's law. The method of obtaining the data is described in the preceding article (pp. 65 to 75), Procedures for Testing Soils for the Determination oft he Subgrade Soil Constants.

PERCENTAGE DETERMINATION

The Bouyoucos hydrometer, like any other, depends upon the density of the suspending medium for its buoyancy. It is calibrated in grams of soil per liter of suspension for assumed conditions of temperature of suspension and specific gravity of soil grains. Consequently, the actual number of grams per liter for any particular case is given directly by the hydrometer reading only if the conditions are identical with those for which the hydrometer was calibrated. For any other conditions suitable corrections must be made. For example, the hydrometer referred to in this discussion has been calibrated for a suspension temperature of 67° F. and a specific gravity of soil particles of 2.65. For temperatures other than 67° F. and specific gravities other than 2.65, the weight of soil remaining in suspension, in grams per liter of suspension, can be obtained by adding a temperature correction to the hydrometer reading and multiplying the sum by a specific gravity correction. The weight thus obtained is expressed as a percentage of the weight of soil originally dispersed. Both weights are given in grams of soil per liter of suspension, but, since the volume of the suspension is ordinarily one liter, the qualifying phrase, "per liter of suspension," is omitted for convenience in the discussion which follows.

When the temperature of the suspension, the specific gravity of the soil particles, and the weight of soil originally dispersed are known, the percentage of soil remaining in suspension for a given hydrometer reading may be determined graphically by the use of a chart laid out on cross-section paper, as illustrated in Figure 1. On this chart ordinates denote hydrometer readings and abscissas denote percentages of soil in suspension. Since the hydrometer is calibrated to give a correct reading in grams per liter when the temperature is 67° F. and the specific gravity is 2.65, these values are used as the base, or standard, to which the graphical corrections are referred. The value 50 grams was chosen as the basic value for the weight of soil originally dispersed. The relation between hydrometer reading and percentage of soil in suspension established by these three standard values is shown as a broken line in Figure 1.

The equation relating the hydrometer reading to the percentage of soil in suspension may be expressed as follows:

Let

R = hydrometer reading.

W = weight of soil originally dispersed per liter of suspension.

w = weight of soil in suspension per liter of suspension.

ΔR = correction to hydrometer reading for variation in temperature from 67° F.

a = correction coefficient for variation in specific gravity from 2.65.

$= \dfrac{2.6500-0.9984}{2.6500} \times \dfrac{G}{G-0.9984}$, where G is the specific gravity of the soil.[1]

P = percentage of originally dispersed soil remaining in suspension.

For the standard values of temperature and specific gravity (67° F. and 2.65),

$$w=R$$

For other values,

$$w=(R+\Delta R)a$$

The percentage of soil in suspension is given by the equation

$$P=\frac{(R+\Delta R)a}{W}\times 100 \qquad (1)$$

This equation may be written

$$P=(R+\Delta R)\times\frac{a}{W}\times 100 \qquad (1)$$

For the basic or standard conditions we have

$$\Delta R=0,\ a=1,\ W=50,$$

so that the basic relation is given by the equation

$$P=\frac{R}{50}\times 100=2R$$

The manner in which the chart is constructed and the method of applying the various corrections are explained in the following paragraphs.

Temperature correction.—In Table 1 are given the values of the correction, ΔR, for variations in temperature from 67° F., obtained experimentally for the hydrometer used as an example in this discussion. If in equation (1) we put $P=0$ we have

$$R=-\Delta R$$

It follows, therefore, that if the line represented by equation (1) should be plotted on the chart (fig. 1) its intercept on the axis of ordinates would be the point 0, $-\Delta R$. In the lower left-hand corner of Figure 1 values of ΔR taken from Table 1 are laid off as ordinates corresponding to values of the temperature from 60° to 90° F. A zero ordinate corresponds to a temperature of 67° F. and positive values of ΔR are plotted downward. By means of this scale the intercept, $-\Delta R$, for any given temperature of suspension, is readily obtained.

It will be noted in equation (1) that the slope of the line is given by the expression $\frac{a}{W}\times 100$. It is evident

[a] Reprinted from PUBLIC ROADS, vol. 12, No. 8, October, 1931.

[1] See preceding article, Procedures for Testing Soils for the Determination of the Subgrade Soil Constants, p. 68.

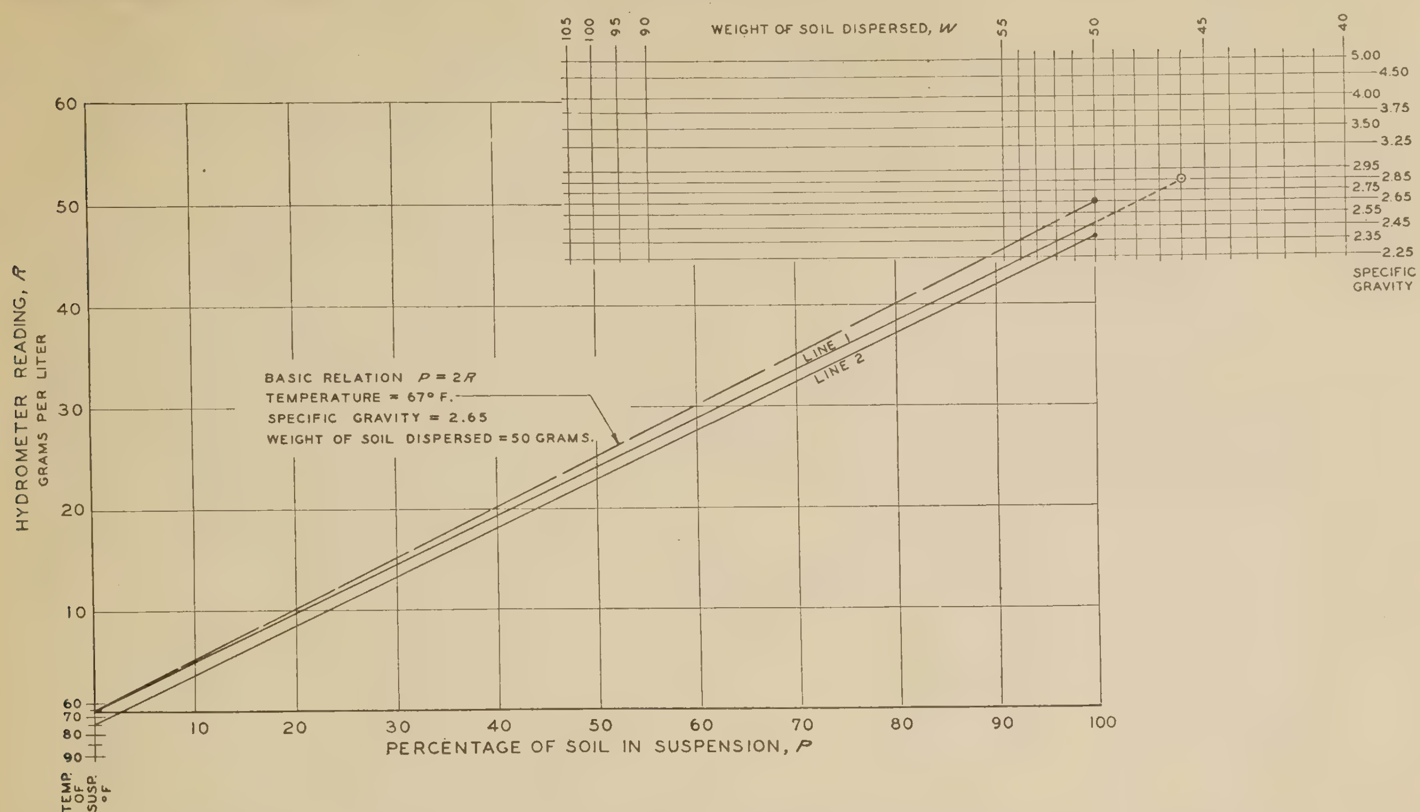


FIGURE 1.—CHART FOR CONVERTING HYDROMETER READINGS TO PERCENTAGES OF ORIGINALLY DISPERSED SOIL REMAINING IN SUSPENSION

TABLE 1.—*Temperature corrections to be added to the hydrometer readings*

Suspension temperature	Correction, ΔR
Degrees F.	*Grams per liter*
60	−0.8
65	−0.3
67	0.0
70	+0.4
75	+1.2
80	+2.2
85	+3.2
90	+4.3

that variations in the relation between P and R caused by variations in the specific gravity and the weight of soil dispersed are cared for by changes in the slope of the line.

Correction for specific gravity.—Assuming the basic values, $\Delta R = 0$, $W = 50$, we have, from equation (1)

$$P = \frac{Ra}{50} \times 100 = 2Ra$$

Setting $P = 100$, we have

$$R = \frac{50}{a} \qquad (2)$$

Equation (2) gives the value of the hydrometer reading at 100 per cent suspension for any given value of specific gravity as indicated by the value of a. In Table 2 are given values of the quantity $\frac{50}{a}$ and also of the quantity $\frac{50}{a} - 50$, the correction which must be added to or subtracted from the basic value 50 to obtain the correct reading for 100 per cent suspension.

In the upper right-hand corner of Figure 1, values of the quantity $R = \frac{50}{a}$ are laid off as ordinates corresponding to values of the specific gravity from 2.25 to 5.00. Thus, for a specific gravity of 2.65 the ordinate is 50; for a value of 5.00 the ordinate is 64.2, etc. When the weight of soil dispersed is 50 grams the slope of the line represented by equation (1) is obtained by drawing a line through the origin intersecting the abscissa $P = 100$ at the value of R corresponding to the given specific gravity.

TABLE 2.—*Values of specific gravity constant, a, $\frac{50}{a}$, and $\frac{50}{a} - 50$ for different values of the specific gravity*

Specific gravity of soil particles, G	Specific gravity constant, a	$\frac{50}{a}$	Hydrometer correction for reading of 50
		Grams per liter	*Grams per liter*
2.25	1.1204	44.6	−5.4
2.35	1.0836	46.1	−3.9
2.45	1.0519	47.5	−2.5
2.55	1.0243	48.8	−1.2
2.65	1.0000	50.0	0.0
2.75	0.9785	51.1	+1.1
2.85	0.9593	52.1	+2.1
2.95	0.9421	53.1	+3.1
3.25	0.8996	55.6	+5.6
3.50	0.8720	57.3	+7.3
3.75	0.8494	58.9	+8.9
4.00	0.8306	60.2	+10.2
4.50	0.8009	62.4	+12.4
5.00	0.7787	64.2	+14.2

Correction for weight of soil dispersed.—Assuming the basic values, temperature = 67° F. and specific gravity of soil = 2.65; i. e., $\Delta R = 0$, $a = 1$, we have, in equation (1)

$$P = \frac{R}{W} \times 100$$

Setting $R = 50$, we have

$$P = \frac{5{,}000}{W} \qquad (3)$$

Equation (3) gives the percentage of soil in suspension for a hydrometer reading of 50 and a weight of dis-

persed soil, W. Values of the quantity $\frac{5,000}{W}$ are given in Table 3 for values of W between 40 and 55 grams and between 90 and 105 grams. The lower interval is for use in the hydrometer analysis of silt and clay soils, the higher interval for use in the case of sandy soils. Values of the quantity $\frac{5,000}{W}$ are plotted as abscissas, in the upper right-hand portion of Figure 1, for indicated values of the weight of soil dispersed, W. Thus, as in Table 3, 50 grams of soil dispersed corresponds with 100 per cent of soil in suspension, 100 grams corresponds with 50 per cent, etc. When the specific gravity of the soil is 2.65, the slope of the line represented by equation (1) is obtained by drawing a line through the origin intersecting the ordinate R=50 at the value of P corresponding to the given value of W.

TABLE 3.—*Values of the quantity* $\frac{5,000}{W}$, *giving the percentage of particles in suspension indicated by a hydrometer reading of 50 for different weights of dry soil originally dispersed. (Specific gravity=2.65; temperature of suspension=67° F.)*

Weight of dry soil dispersed, W, per liter of suspension	Percentage in suspension, P, for hydrometer reading of 50	Weight of dry soil dispersed, W, per liter of suspension	Percentage in suspension, P, for hydrometer reading of 50
Grams		*Grams*	
40.0	125.0	90.0	55.6
41.0	122.0	91.0	54.9
42.0	119.1	92.0	54.3
43.0	116.3	93.0	53.8
44.0	113.7	94.0	53.2
45.0	111.1	95.0	52.6
46.0	108.7	96.0	52.1
47.0	106.4	97.0	51.5
48.0	104.2	98.0	51.0
49.0	102.0	99.0	50.5
50.0	100.0	100.0	50.0
51.0	98.0	101.0	49.5
52.0	96.2	102.0	49.0
53.0	94.3	103.0	48.5
54.0	92.6	104.0	48.1
55.0	90.9	105.0	47.6

For given values of temperature of suspension, specific gravity of soil, and weight of soil originally dispersed per liter of suspension, the line represented by equation (1), giving the relation between hydrometer reading and percentage of soil in suspension, is laid out on the chart of Figure 1 by the following process:

1. A straightedge is placed on the chart so as to intersect the origin and a point whose abscissa corresponds with the given weight of soil dispersed and whose ordinate corresponds with the given specific gravity, as indicated by the scales shown in the upper right-hand corner of Figure 1.

2. The straightedge is then moved parallel to its original position until it intersects the axis of ordinates at a point corresponding to the given temperature of suspension, as indicated by the temperature scale in the lower left-hand corner of Figure 1. A line is then drawn which represents the required relation between P and R.

To illustrate this operation, let us assume that 46.0 grams of soil having a specific gravity of 2.85 were dispersed and that the temperature of the suspension remained constant at 75° F. Line 1 in Figure 1, intersecting the origin and a point whose coordinates correspond with 46.0 grams of soil dispersed and a specific gravity of 2.85, represents the first position of the straightedge. Line 2, which intersects the axis of ordinates at a point corresponding to 75° F., gives the desired relation between P and R.

Values given by line 2 may be checked by computations based on equation (1). From Tables 1 and 2 we obtain the values, $\Delta R = +1.2$ and $a = 0.9593$. Substituting in equation (1), we have

$$P = (R + 1.2)\,\frac{0.9593}{46.0} \times 100$$

$$= (R + 1.2)\ 2.086$$

If $P=0$, $R=-1.2$. If $R=32.0$, $P=69.3$ These values will be found to check with those given by line 2.

DETERMINATION OF GRAIN SIZE

The second distinct operation in the graphical solution of the data furnished by the hydrometer analysis consists of determining the maximum grain size in suspension at any given time from Stokes's law, which is expressed by the formula

$$d = \sqrt{\frac{30nL}{980\,(G - G_1)\,T}} \text{-------------} (4)$$

Where
- d = maximum grain diameter in millimeters.
- n = coefficient of viscosity of the suspending medium, in poises.
- L = distance in centimeters through which soil particles settle.
- T = time in minutes, period of sedimentation.
- G = specific gravity of soil particles.
- G_1 = specific gravity of the suspending medium.

In order that Stokes's law may serve to disclose the diameter of the soil particles it is necessary to know the distance through which these particles fall in a given time. As has been explained previously in the report, Procedures for Testing Soils for the Determination of the Subgrade Soil Constants, the distance, L, through which the soil particles are assumed to settle, in the determination of the grain size by means of the Bouyoucos hydrometer, equals 0.42 of the total distance between the surface of the suspension and the elevation of the bottom of the hydrometer.

When the hydrometer reading, the temperature of the suspension, the period of sedimentation, and the specific gravity of the soil particles are known, the particle diameter may be determined graphically by the use of a chart constructed as shown in Figure 2. The chart is plotted on semilogarithmic cross-section paper, on which the ordinates denote hydrometer reading and the abscissas denote period of sedimentation.

In the preceding article (p. 69) variations in L, n, and G (equation 4) were cared for by means of correction coefficients applied to values of d computed on the basis of the standard conditions, temperature = 67° F., G = 2.65, and L = 32.5 centimeters. The graphical method here described does not involve the use of the standard value, L = 32.5 centimeters. Values of L as determined by actual measurements of the hydrometer in use are substituted in the equation, and recourse to a correction factor is eliminated. The manner in which variations of L, n, and G are cared for is described in the following paragraphs.

Variations in the factor L.—Solving equation (4) (Stokes's law) for T, the period of sedimentation, we have

$$T = \frac{30nL}{980\,(G - G_1)\,d^2} \text{-------------} (5)$$

Values of T as a function of L, the distance through which the soil settles, and d, the maximum diameter of

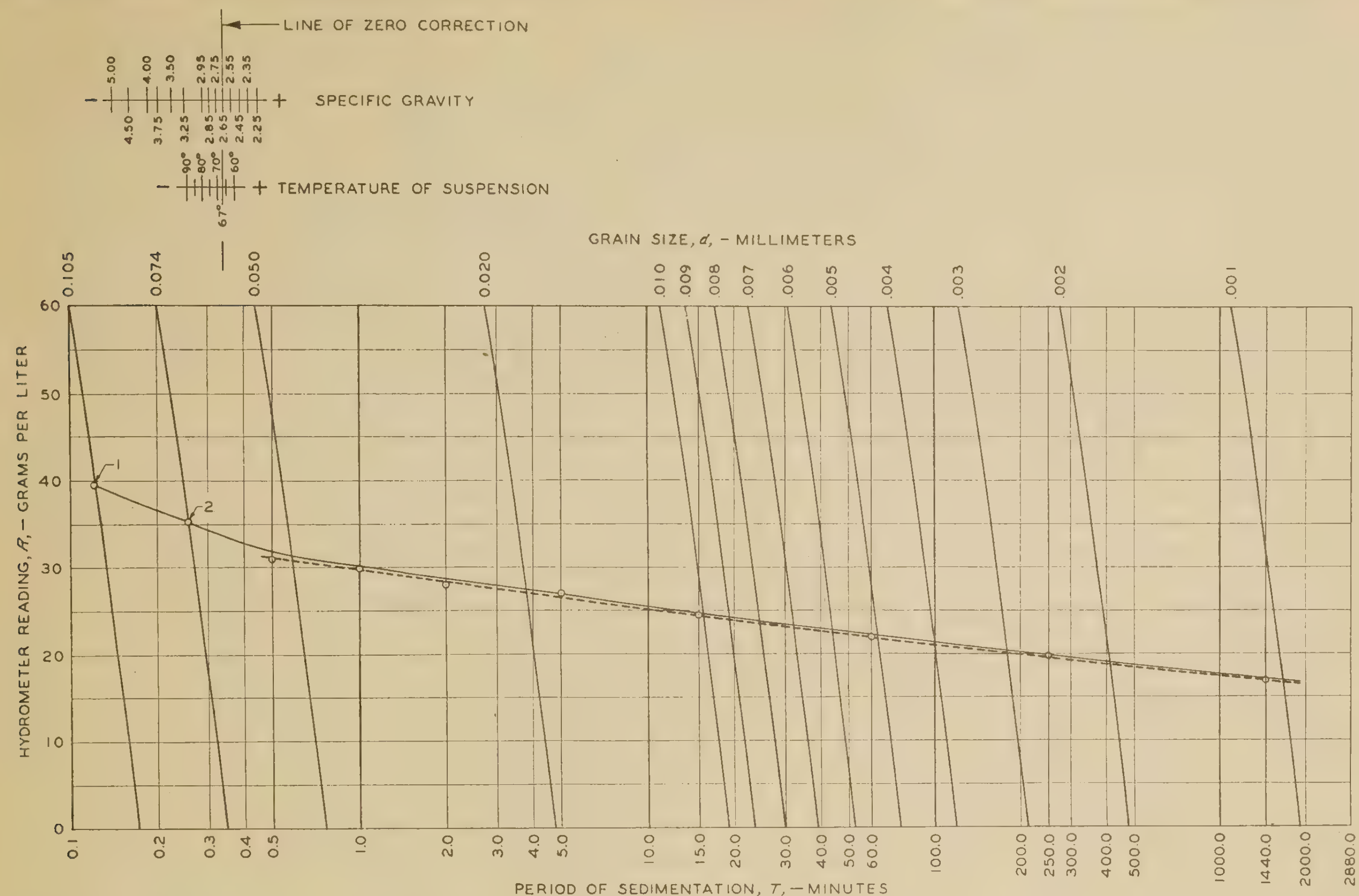


FIGURE 2.—CHART FOR OBTAINING FROM TEST DATA THE HYDROMETER READINGS CORRESPONDING TO SPECIFIC GRAIN SIZES

TABLE 4.—*Time of sedimentation as a function of grain diameter, d, and hydrometer reading, R. (Specific gravity=2.65; temperature of suspension=67° F.)*

Hydrometer reading, R	L	Time, T, in minutes for the following diameters—													
		0.105 millimeters	0.074 millimeters	0.050 millimeters	0.020 millimeters	0.010 millimeters	0.009 millimeters	0.008 millimeters	0.007 millimeters	0.006 millimeters	0.005 millimeters	0.004 millimeters	0.003 millimeters	0.002 millimeters	0.001 millimeters
Grams per liter	*Centimeters*														
0	10.00	0.171	0.345	0.756	4.73	18.9	23.3	29.5	38.6	52.5	75.6	118.0	210	473	1,891
10	9.25	.159	.319	.700	4.37	17.5	21.6	27.3	35.7	48.6	70.0	109.0	194	437	1,749
20	8.47	.145	.292	.641	4.00	16.0	19.8	25.0	32.7	44.5	64.1	100.0	178	400	1,601
30	7.78	.133	.269	.588	3.68	14.7	18.2	23.0	30.0	40.9	58.8	91.9	163	368	1,471
40	7.07	.121	.244	.535	3.34	13.4	16.5	20.9	27.3	37.1	53.5	83.5	149	334	1,337
50	6.37	.109	.220	.482	3.01	12.0	14.9	18.8	24.6	33.5	48.2	75.3	134	301	1,204
60	5.75	.099	.199	.435	2.72	10.9	13.4	17.0	22.2	30.2	43.5	67.9	121	272	1,087

soil particle remaining in suspension, are given in Table 4. The values of L to be used were determined by measurement for hydrometer readings from 0 to 60. The following quantities were assumed to be constant:

$n=0.0102$, the coefficient of viscosity of water at 67° F.
$G=2.65$.
$G_1=0.9984$, the density of water at 67° F.

The significance of Table 4 may be made clear by consideration of specific values. Thus, if the hydrometer reading, after a sedimentation period of 0.269 minutes, has the value 30, the maximum diameter of soil particles remaining in suspension is 0.074 millimeters. If, on the other hand, it takes 30 minutes for the hydrometer reading to reach the value 30, the maximum diameter of soil particles is 0.007 millimeters

The values given in Table 4 are plotted in Figure 2 as curves showing the relation between hydrometer reading and period of sedimentation for different values of d, the maximum grain size. The relation between hydrometer reading and maximum grain size for a hydrometer analysis in which the temperature of suspension was 67° F. and the specific gravity of the soil was 2.65 may be obtained by plotting hydrometer readings against periods of sedimentation, as given by the analysis, on the chart of Figure 2. Intersections of the curve so determined with the grain-diameter curves give the hydrometer readings corresponding to specific values of maximum diameter of soil in suspension.

Variations in temperature and specific gravity.—Equation (5) may be written

$$T=\frac{30L}{980d^2}\times n\times\frac{1}{G-G_1} \qquad (5)$$

For a temperature of 67° F. and a specific gravity of 2.65, we have

$$T_o=\frac{30L}{980d^2}\times 0.0102\times\frac{1}{2.6500-0.9984}$$

so that

$$\frac{T}{T_o}=\frac{n}{0.0102}\times\frac{1.6516}{G-G_1}$$

Let

$$C_n=\frac{n}{0.0102}$$

and

$$C_G=\frac{1.6516}{G-G_1}$$

$$=\frac{1.65}{G-1}\text{ (approximately).}$$

Then

$$T=T_o\times C_n\times C_G,$$

and

$$\text{Log } T=\text{Log } T_o+\log C_n+\log C_G \text{-------(6)}$$

The grain-diameter curves plotted in Figure 2 give correct values of T, the period of sedimentation, for the standard conditions, temperature = 67° F. and G = 2.65. To obtain correct values for other temperatures and specific gravities, it is necessary to multiply all values of T by the factor $C_n\times C_G$. Since the scale of T is logarithmic, it is evident that the correction can be applied by displacing the entire system of curves horizontally a distance equal to the algebraic sum of the corrections, $\log C_n+\log C_G$. If this sum is positive, the curves should be shifted to the right; if negative, to the left.

Values of the viscosity coefficient of water, n, the temperature correction $C_n=\frac{n}{0.0102}$, and $\log_{10} C_n$ are given in Table 5 for temperatures varying from 60° to 90° F. Values of C_n are plotted against temperature of suspension in the upper left-hand corner of Figure 2 to the same logarithmic scale as that to which the period of sedimentation is plotted on the main chart. The reference line (temperature = 67° F., C_n = 1.00, $\log C_n$ = 0) may be taken at any convenient point. Distances along this correction scale may be laid off directly from the logarithmic scale by the use of dividers. For construction purposes the actual lengths may be computed accurately by multiplying each value of $\log C_n$ by the length of one cycle on the chosen logarithmic scale.

TABLE 5.—*Values of the viscosity coefficient,*[1] *n, of water at various temperatures, of the coefficient* $C_n=\frac{n}{0.0102}$, *and of* $\log_{10} C_n$.

Temperature	n	Coefficient C_n	$\text{Log}_{10} C_n$
° F.	Poises		
60	0.0112	1.10	0.04139
65	.0105	1.03	0.01284
67	.0102	1.00	0.00000
70	.00978	0.959	−0.01818
75	.00917	0.899	−0.04624
80	.00861	0.844	−0.07366
85	.00810	0.794	−0.10018
90	.00764	0.749	−0.12552

[1] Smithsonian Physical Tables, seventh revised edition, 1921.

Similarly, values of C_G and $\log_{10} C_G$ are given in Table 6 for specific gravities varying from 2.25 to 5.00, and a logarithmic scale of these corrections is shown above the temperature correction scale in Figure 2.

As stated above, the grain-diameter curves may be corrected for given values of temperature and specific gravity by horizontal displacement in the direction indicated by the sign of the correction. To plot a point representing a given hydrometer reading and a given period of sedimentation, it is necessary to apply the correction in the opposite direction, i. e., positive to the left. The relative position of the point with respect to the curves is then the same as if the curves had been shifted to the right (or to the left, in the case of a negative correction).

TABLE 6.—*Values of the specific gravity correction,* $C_G=\frac{1.65}{G-1}$, *and of* $\log_{10} C_G$, *for values of the specific gravity, G, from 2.25 to 5.00*

Specific gravity, G	Coefficient, C_G	$\text{Log}_{10} C_G$
2.25	1.320	0.1206
2.35	1.222	0.0872
2.45	1.138	0.0561
2.55	1.064	0.0272
2.65	1.000	0.0000
2.75	0.943	−0.0256
2.85	0.892	−0.0497
2.95	0.846	−0.0726
3.25	0.733	−0.1347
3.50	0.660	−0.1805
3.75	0.600	−0.2218
4.00	0.550	−0.2596
4.50	0.471	−0.3266
5.00	0.412	−0.3846

METHOD OF USING CHARTS DESCRIBED

In Table 7 are given the data obtained in the hydrometer analysis of a soil sample. The following paragraphs describe the steps involved in using the charts of Figures 1 and 2 to develop from this material the data on which the grain-size accumulation curve is based. The results thus obtained are given in Table 8.

TABLE 7.—*Sieve and hydrometer analysis of sample 5,394X*

A.—GENERAL DATA

Weight of air-dried sample, grams	49.0
Weight of dry soil dispersed, grams	46.0
Specific gravity	2.85
Plasticity index	18.0

B.—HYDROMETER TEST DATA[1]

Date tested	Time observed	Temperature	Hydrometer reading	Period of sedimentation
		°F.	Grams per liter	Minutes
June 25, 1930	9.30 a. m.			0
Do	9.30.5 a. m.	76	31.0	0.5
Do	9.31 a. m.	75	30.0	1
Do	9.32 a. m.	75	28.0	2
Do	9.35 a. m.	75	27.0	5
Do	9.45 a. m.	75	24.5	15
Do	10.00 a. m.	74	24.0	30
Do	10.30 a. m.	74	22.0	60
Do	1.40 p. m.	74	20.0	250
June 26, 1930	9.30 a. m.	73	17.0	1,440

C.—SIEVE ANALYSIS

Fraction	Weight	Percentage of dispersed sample
	Grams	
Retained on No. 10		0
Passing No. 10, retained on No. 20	0.98	2.13
Passing No. 20, retained on No. 40	0.80	1.74
Passing No. 40, retained on No. 60	0.90	1.96
Passing No. 60, retained on No. 140	4.32	9.39
Passing No. 140, retained on No. 200	3.80	8.26

[1] Volume of suspension, 1 liter.

1. On tracing paper placed over Figure 1, line 2 is constructed by the method previously described, for a specific gravity, *G*, of 2.85, a weight of dry soil dispersed, *W*, of 46.0 grams, and a temperature of suspension of 75° F., as indicated in Tables 7, A and 7, B. While some of the temperatures listed in Table 7, B vary slightly from 75° F., the temperature throughout the test may be assumed as being 75° F.

2. The percentages of material retained on each of the sieves in the sieve analysis are computed and listed in Table 7, C. From these data the percentage of material smaller than each of the sieve sizes is computed and recorded in the last column of Table 8.

3. On a piece of tracing paper placed over Figure 2 a curve of time against hydrometer reading is plotted, as shown by the broken line in Figure 2, from the observed data as recorded in Table 7, B. The intersections of this curve with the grain-diameter curves give the hydrometer readings corresponding to the times of sedimentation indicative of the grain sizes represented by the grain-diameter curves, uncorrected for variation of the specific gravity of the soil particles in suspension and the temperature of the suspension from the standard values, 2.65 and 67° F., respectively.

4. The tracing paper is shifted horizontally through a distance equal to the algebraic sum of the temperature and specific gravity corrections, which in this case, for a temperature of 75° F. and a specific gravity of 2.85, is to the right. It should be noted that moving the tracing paper upon which the curve of time against hydrometer reading has been plotted produces the same effect as moving the grain-diameter curves in the opposite direction. It is for this reason that a point representing the algebraic sum of the specific gravity and temperature corrections, easily obtained by means of a pair of dividers, is laid off from the line of zero correction in the direction corresponding to the sign of that sum and the tracing paper moved until this point coincides with the line of zero correction.

This curve in its shifted position, shown as a full line in Figure 2, gives correctly the hydrometer readings corresponding to the grain sizes indicated by the grain-diameter curves.

5. The hydrometer readings corresponding to the percentages of material passing the No. 140 and the No. 200 sieves as recorded in Table 8 are obtained from line 2, Figure 1. In this case the values are 84.78 per cent and 76.52 per cent, respectively, and their computed hydrometer readings are 39.5 and 35.3. These hydrometer readings are plotted on the grain-diameter curves (fig. 2) corresponding to their respective sieve sizes with the tracing paper in its shifted position. These points are shown as points 1 and 2, Figure 2.

6. The two points are then connected with the curve previously drawn (full line, fig. 2) in order to tie in the results given by the sieve analysis with those given by the hydrometer analysis.

7. The hydrometer readings corresponding to the grain sizes tabulated in Table 8 are obtained from the curve (full line, fig. 2). Thus the hydrometer reading corresponding to a grain diameter of 0.05 millimeter is 32.0 and the hydrometer reading corresponding to a grain diameter of 0.002 millimeter is 19.3.

8. For these hydrometer readings the corresponding percentages of soil in suspension are obtained from line 2 (fig. 1). Thus a hydrometer reading of 32.0 indicates 69.3 per cent of soil in suspension and a hydrometer reading of 19.3 indicates 42.8 per cent of soil in suspension. These values are recorded in Table 8 and used in plotting the soil accumulation curve, which is shown in Figure 3. From this curve the percentage of soil smaller than any specific diameter may be obtained.

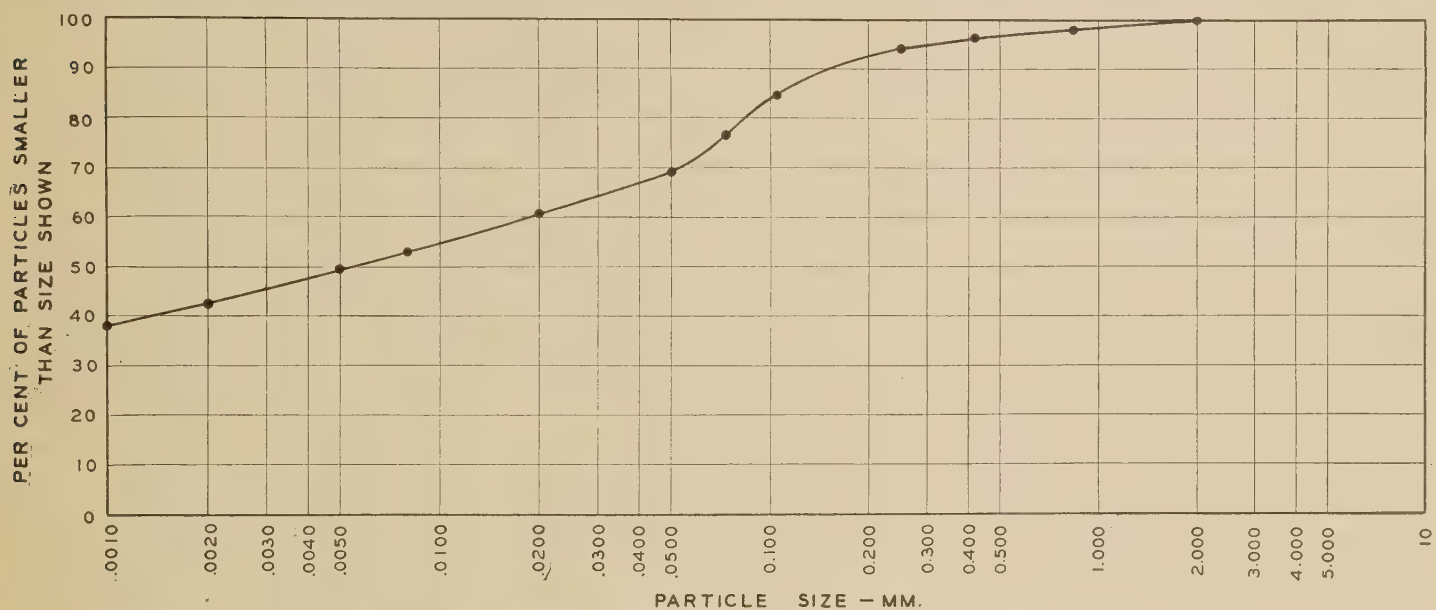


FIGURE 3.—GRAIN-SIZE ACCUMULATION CURVE FOR SOIL SAMPLE 5,394X

TABLE 8.—*Grain-size accumulation data for soil sample 5,394X*

Sieve No.	Maximum grain size of fraction	Hydrometer reading	Percentage of total sample
	Millimeters	*Grams per liter*	
10	2.0	------------	100.00
20	0.84	------------	97.87
40	0.42	------------	96.13
60	0.25	------------	94.17
140	0.105	[1] 39.5	84.78
200	0.074	[1] 35.3	76.52
	0 05	32.0	69.3
	0.02	28.0	61.0
	0.008	24.4	53.4
	0.005	22.5	49.4
	0.002	19.3	42.8
	0.001	17.2	38.4

[1] Computed.

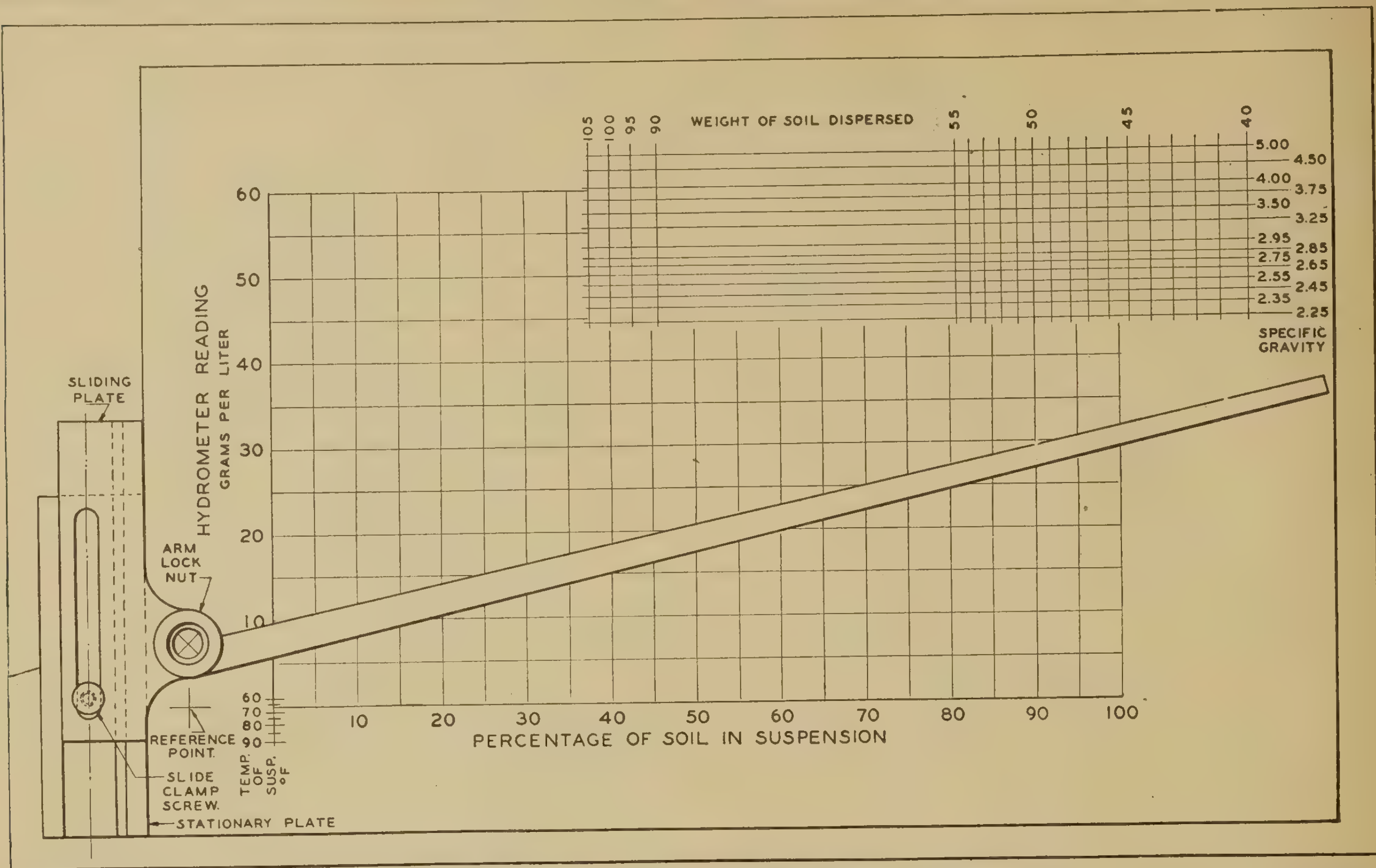


Figure 4.—Chart and Protractor for Converting Hydrometer Readings to Percentages of Originally Dispersed Soil Remaining in Suspension

SPECIAL APPARATUS ASSISTS IN PERFORMING THE GRAPHICAL SOLUTION

The foregoing procedure was described in detail in order to explain clearly the basis of the various operations. The procedure used in the laboratory of the Bureau of Public Roads is the same as that described above in essentials, but is somwhat simplified by the use of special apparatus.

The apparatus which assists in determining the relation between the hydrometer reading and the percentage of soil in suspension is shown in Figure 4. It consists of a chart containing scales similar to those shown in Figure 1 and an adjustable protractor which slides on a fixed base. Both the protractor and the chart are mounted on a drawing board 12 inches wide and 16 inches long.

The scales for hydrometer reading, percentage of soil in suspension, temperature, and specific gravity are constructed exactly like and in the same position as the same scales in Figure 1. A reference point for the fulcrum of the protractor arm is located 1 inch to the left of the origin. To compensate for this displacement, the scale for weight of soil dispersed is shifted 1 inch to the left of the position shown in Figure 1. This displacement was necessitated by the fact that if the reference point were at the origin the fulcrum of the protractor would interfere with reading the scales in the lower left-hand corner of the chart.

The protractor is operated in the following manner: The slide clamp screw is loosened and the protractor is moved vertically until the intersection of the cross hairs in the transparent center of the fulcrum of the protractor is directly over the reference point. The slide clamp screw is then tightened and the arm lock nut is loosened. The protractor arm is rotated until its reading edge passes through a point whose abscissa corresponds to the given weight of soil displaced and whose ordinate corresponds to the given specific gravity. The arm lock nut is then tightened, the slide clamp screw is loosened, and the protractor is moved vertically until the reading edge of the arm intersects the temperature scale at the desired point. The slide clamp screw is then tightened. In this position the reading edge of the protractor arm, corresponding in position to line 2 (fig. 1), will give correctly the percentage of soil particles in suspension indicated by the hydrometer readings.

The apparatus which assists in determining the grain size is illustrated in Figure 5. It consists of a chart similar to that shown in Figure 2, but without the temperature and specific gravity scales, and a sliding carriage which serves to shift the tracing paper, all mounted on a drawing board 16 by 21 inches in size. The tracing paper is held in the sliding carriage by two clamps. The sliding of the carriage is controlled by means of a thumbscrew and thrust yoke located at the upper right-hand corner of the board.

The specific gravity scale is scribed on the carriage exactly as it was plotted in Figure 2, and the temperature scale is scribed on a fixed metal guide adjacent to the specific gravity scale. The scale values are the same as were used in the construction of the temperature scale (fig. 2), but positive values are plotted to the left and negative values to the right of the zero correction line. This is necessary because the specific gravity scale moves with the tracing paper but the temperature scale remains fixed. The position of these scales on the apparatus is shown in Figure 5. The chart containing the grain-size curves is held in place on the drawing board by means of two clamps, as shown.

This apparatus is operated in the following manner: The tracing paper is inserted in the slot in the movable carriage provided for it and the clamps are tightened. By means of the adjusting screw the position of the described. The tracing paper is then displaced horizontally by moving the slide carriage until the two correction scales are in such a position that the given specific gravity coincides with the given temperature.

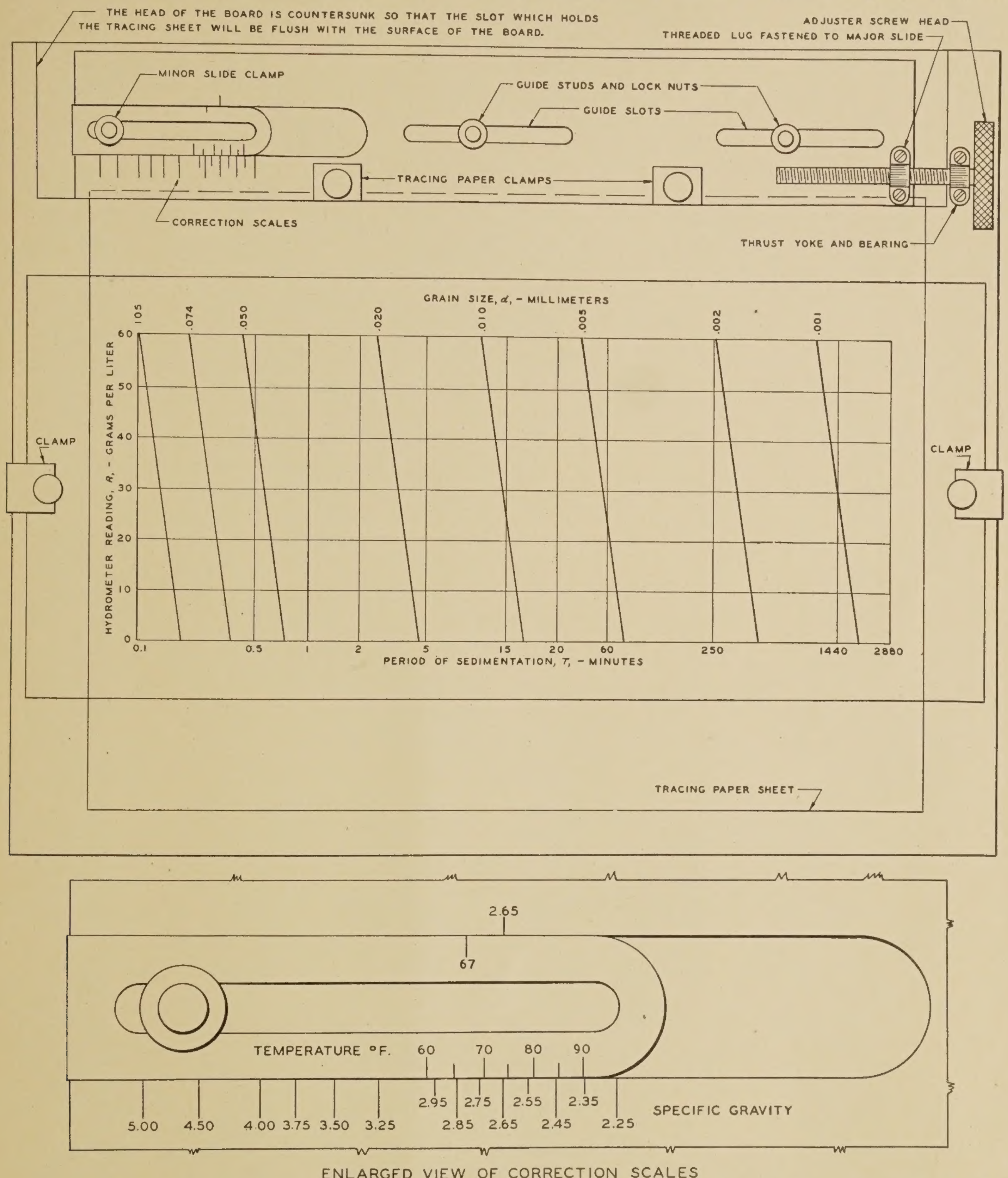


Figure 5.—Chart and Mechanical Device for Obtaining from Test Data the Hydrometer Readings Corresponding to Specific Grain Sizes

carriage is adjusted so that a specific gravity of 2.65 on the specific gravity scale coincides with a temperature of 67° F. on the temperature scale. A curve of time of sedimentation against hydrometer reading is plotted on the tracing paper in the manner previously The curve of time of sedimentation against hydrometer reading will then be in the correct position to give the hydrometer readings corresponding to the grain diameters for which the information is desired.

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